Empirical Deck for Phased Construction & Widening

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FDOT Engineering Specialist: **David Wagner**

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Outline

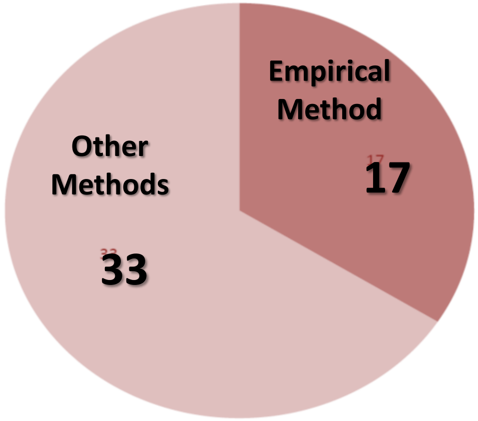
- Introduction
- Background & Justification
- FE
- CONSPAN Design
- Mathcad model
- Experimental work (beams, formwork, steel, instrumentation, concrete, and testing)

Introduction

The AASHTO-LRFD Specifications include two methods of deck design.

1. The first method is called the traditional design method (S4.6.2.1) and is typically referred to as the equivalent strip method.

2. The second is called the empirical design method (S9.7.2).



(Nielsen, et al., 2010)

Justification for the proposed research

- All deck slabs are required to be designed according to *AASHTO's Traditional Design Method* (9.7.3). The traditional design method typically results in a higher ratio of steel than the empirical method in the final stage.

- Currently, the *empirical design method* for deck slabs as per *AASHTO LRFD* 9.7.2.4 is not allowed in Florida as per *Structures Design Guidelines (SDG)* 4.2.4. According to the SDG the empirical design method is not permitted because of the potential for future widening or phased construction and associated traffic control impact in order to comply with AASHTO LRFD 9.7.2.4. There is potential for cost savings if economical methods can be completed to ensure that the empirical design will work during phased construction and/or widening.

NYSDOT Bridge Manual

5.1.5.1 Isotropic Decks The design of isotropic reinforced decks is based on empirical results that show reinforced concrete bridge decks develop an arching action between girders and fail in punching shear rather than flexure when subjected to loads that are significantly higher than factored design loads. Isotropic reinforced decks have lighter reinforcement than traditionally reinforced decks and use equal reinforcement transversely and longitudinally in both top and bottom mats. Reinforcement in deck overhangs is designed for flexure the same as for conventional decks.

The maximum center-to-center spacing of the girders is 11 ft. and the minimum spacing is 5 ft.

The minimum overhang, measured from the centerline of the fascia girder to the fascia, is 2’-6”. If a concrete barrier composite with the deck is used, the minimum overhang is 2’-0”.

Arching or Compressive Membrane Action in RC slabs

Punching Shear Failure

- Punching shear failure normally occurs in reinforced concrete slabs subjected to concentrated loads and particularly in concrete bridge decks due to development of an internal arching action within the system.

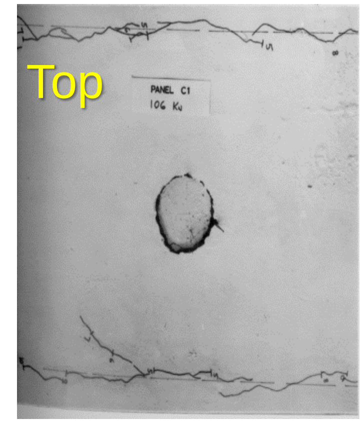
- The governing failure mode for concrete bridge decks is not flexure. The primary structural action by which these slabs resist concentrated wheel loads is not flexure but an internal membrane stress state referred to as internal arching.

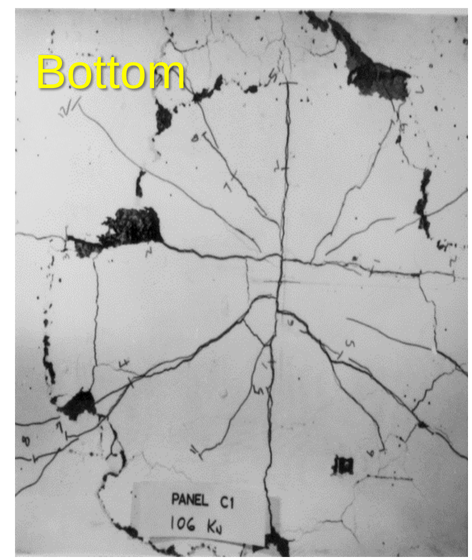
- **Due to typical high rigidity of bridge girders and high thickness-to-span ratio of typical bridge deck slabs, the load mechanism developed into the slab creates an arch action rather than flexural behavior mechanism to resist the applied wheel loads.**

- The bottom reinforcements of the bridge deck slab act as ties for the arch action mechanism rather than flexural reinforcement for the positive moments

- Using flexural design method usually led to artificial high levels of steel reinforcement

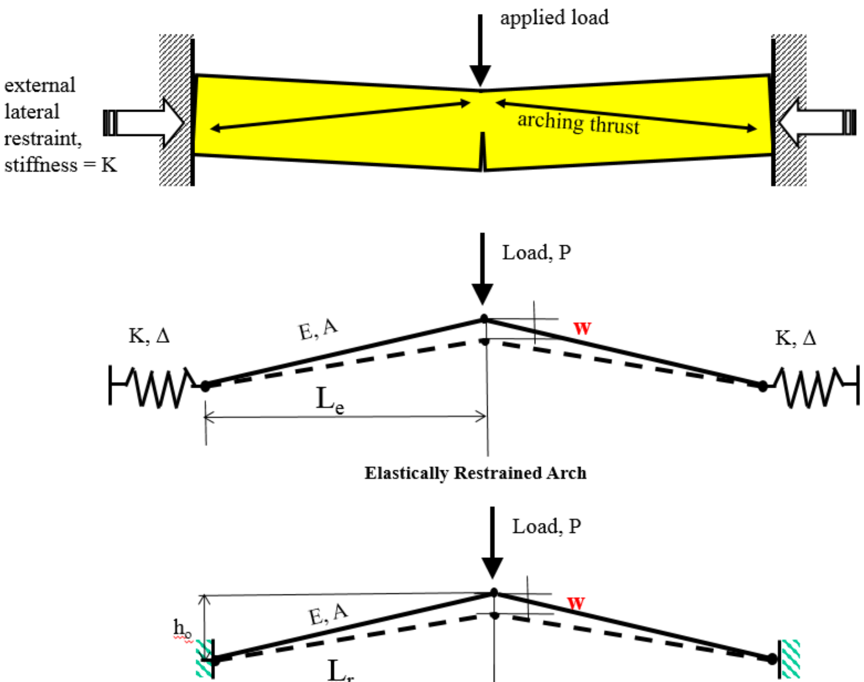
Top surface crack pattern of punching failure zone in model bridge deck test (Kirkpatrick, 1982)



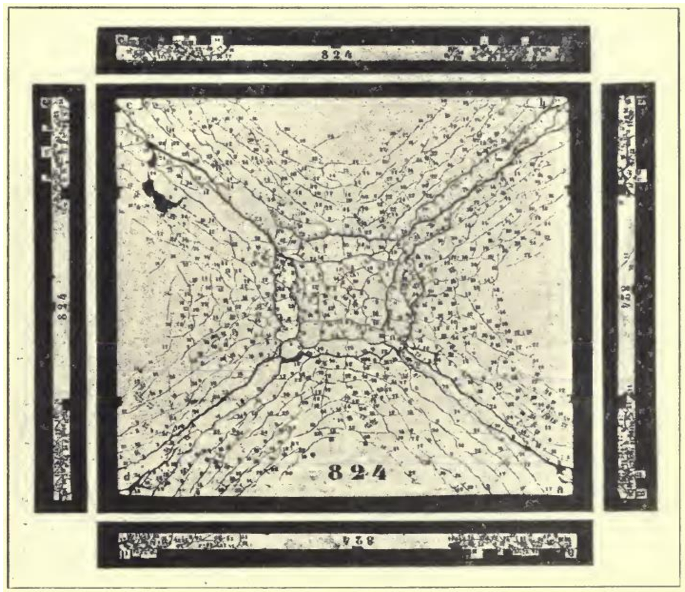


Bottom surface crack pattern of punching failure zone in model bridge deck test (Kirkpatrick, 1982)

Arching or Compressive Membrane Action in RC slabs



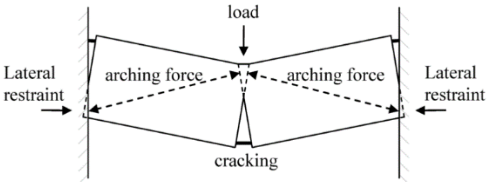
Arching action and three hinged arch analogy (Rankin 1982)

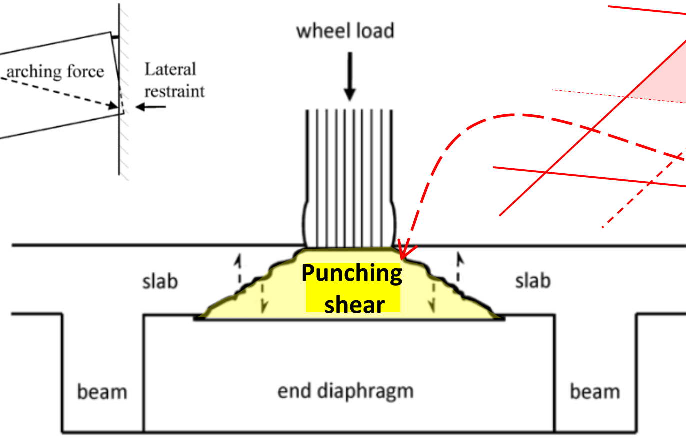


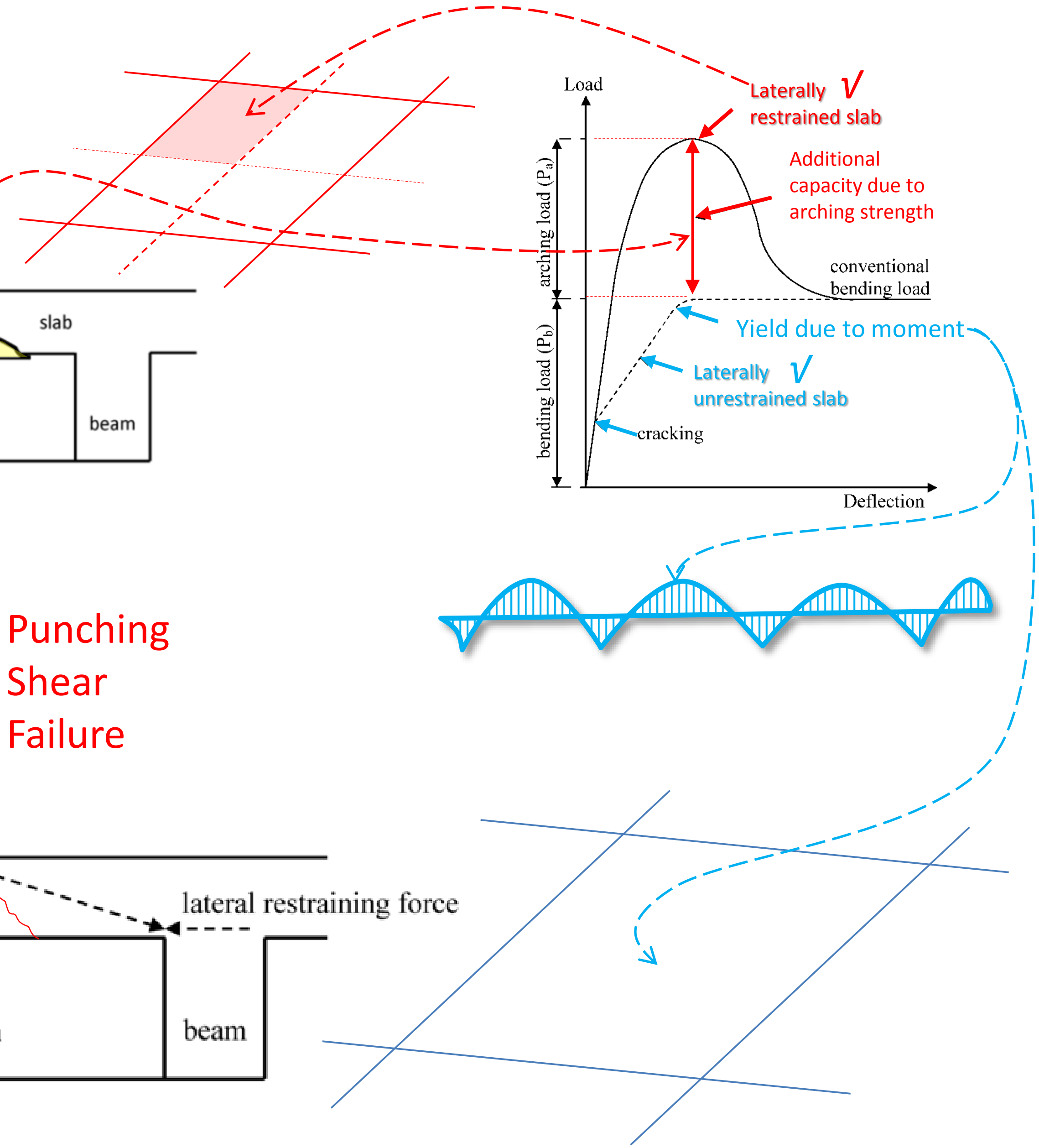
Bottom of square slab of 200 cm. span, tested by Bach and Graf

Arching or Compressive Membrane Action in RC slabs

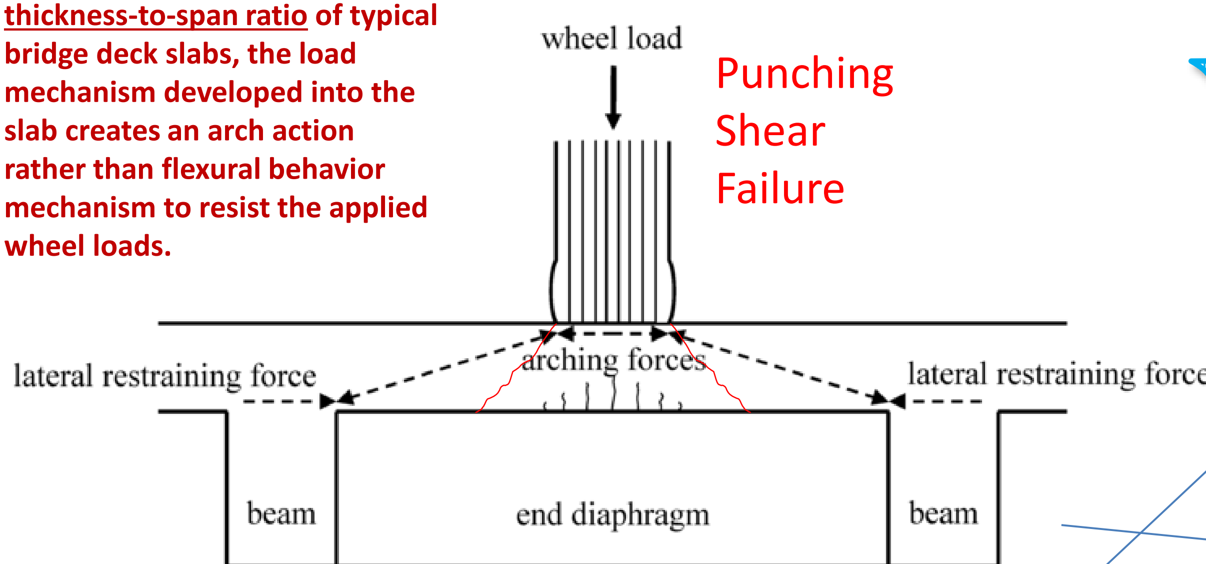
Idealization of arching action forces in laterally restrained slab





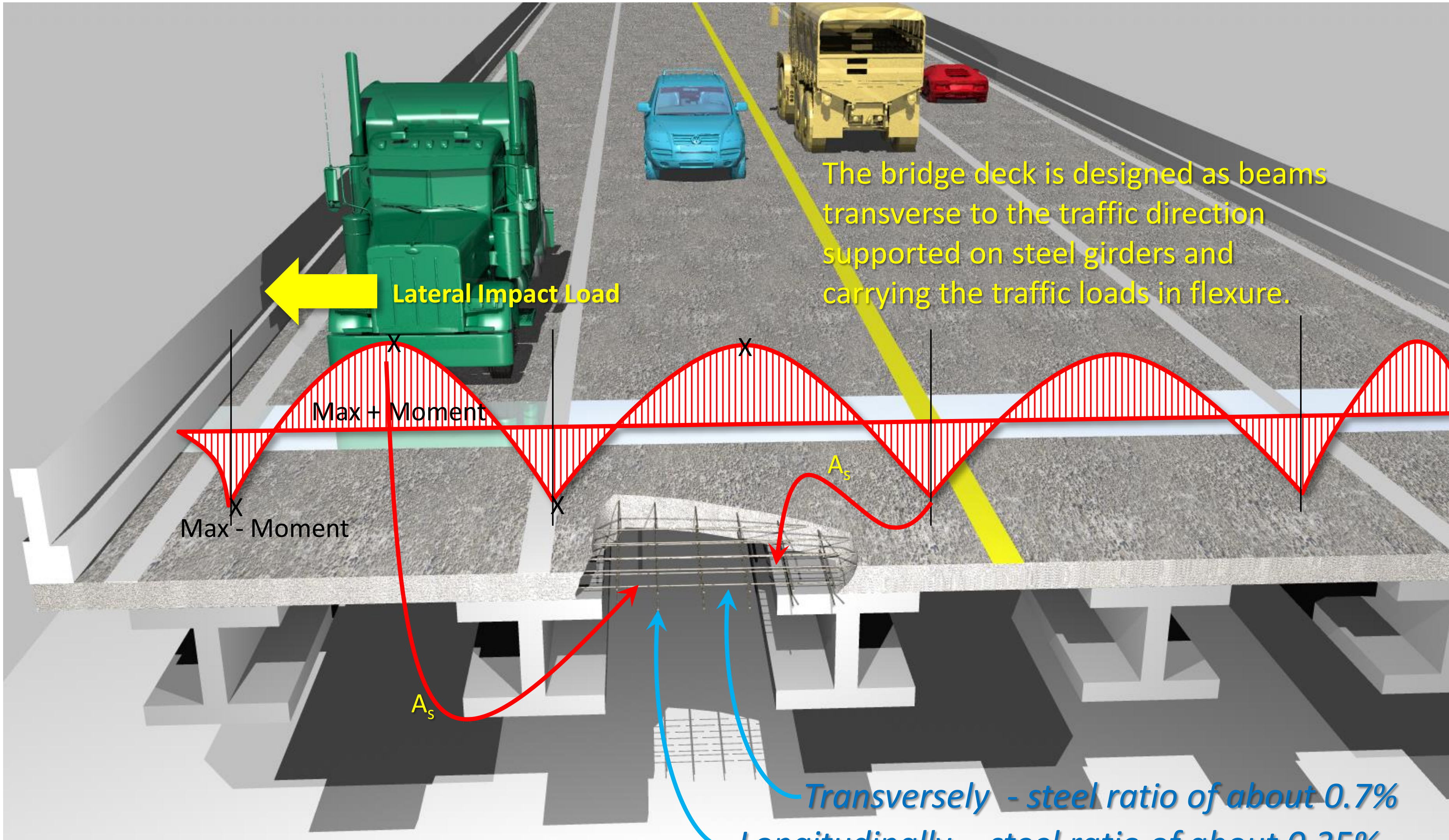


Due to typical high rigidity of bridge girders and <u>high thickness-to-span ratio</u> of typical bridge deck slabs, the load mechanism developed into the slab creates an arch action rather than flexural behavior mechanism to resist the applied wheel loads.



Punching Shear Failure

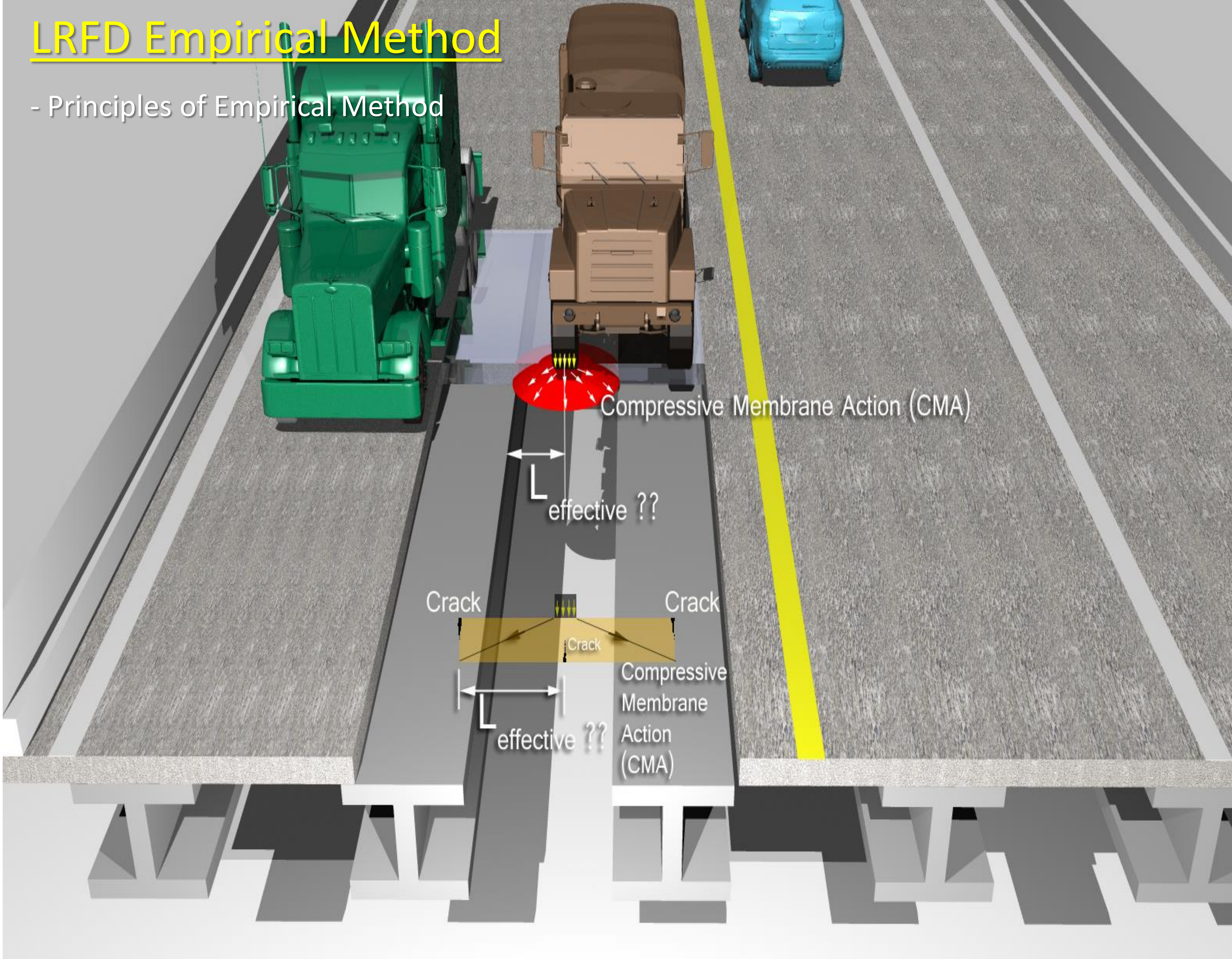
LRFD Traditional Method (Equivalent Strip Method)



Orthotropic Deck

LRFD Empirical Method

- Principles of Empirical Method

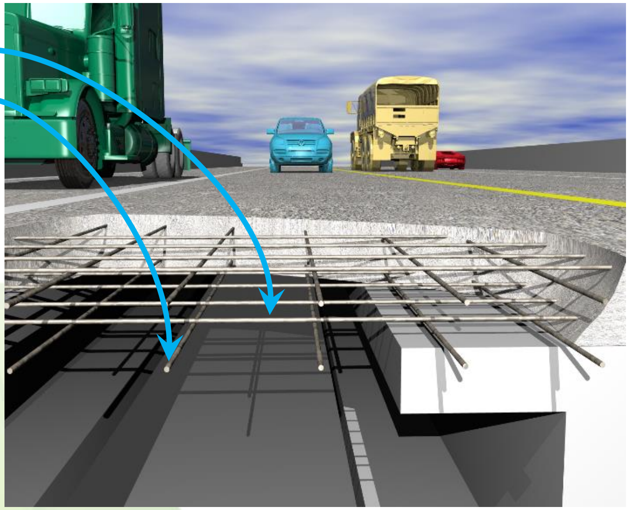


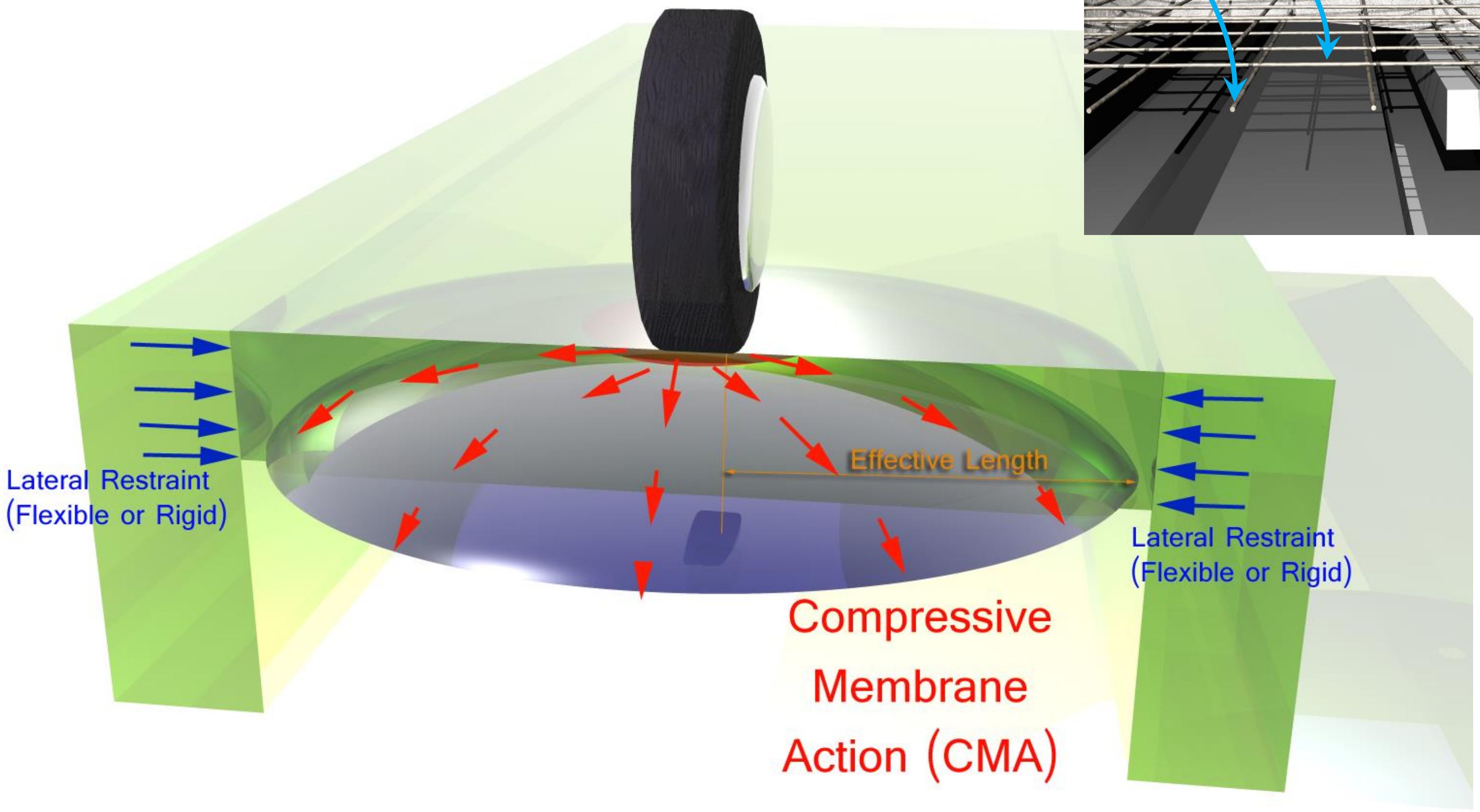
LRFD Empirical Method

Transversely - steel ratio of about 0.3%

Longitudinally - steel ratio of about 0.3%

Isotropic Deck



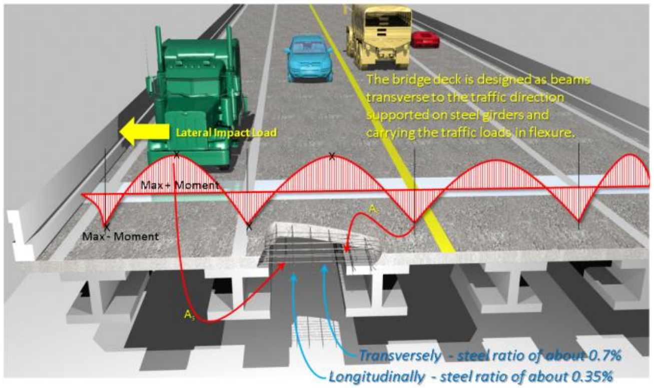


Wheel Load Transfer

The added strength gained from this "arching action" allows for a reduction in reinforcing steel requirements

Florida Requirements

LRFD Traditional Method (Equivalent Strip Method)



Orthotropic Deck

- ❑ In Florida, all deck slabs are required to be designed according to *AASHTO's Traditional Design Method* (9.7.3).

The traditional design method typically results in a higher ratio of steel than the empirical method in the final stage.

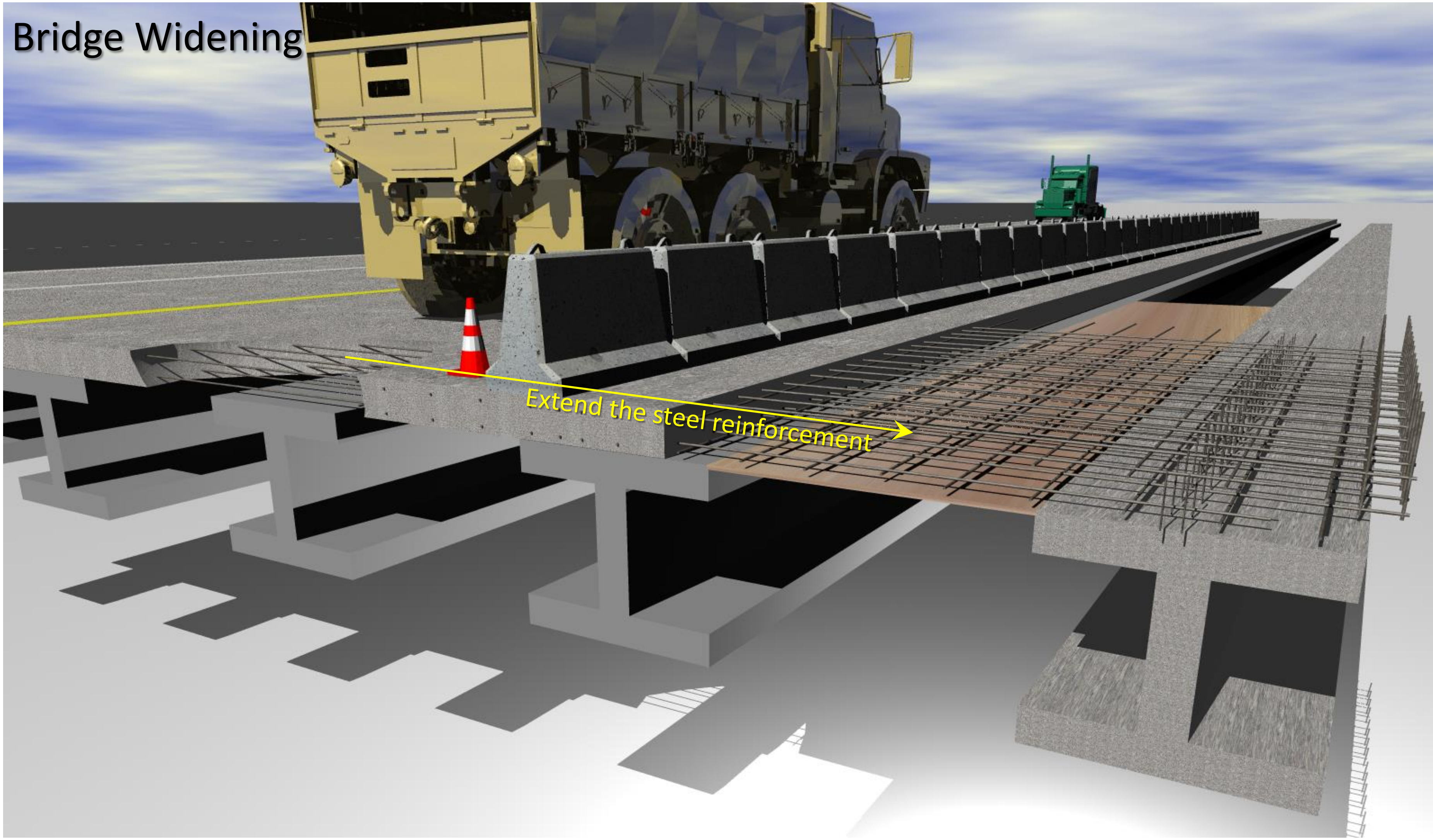
FDOT Structures Design Guidelines — Topic No. 625-020-018

4 - Superstructure - Concrete — January 2010

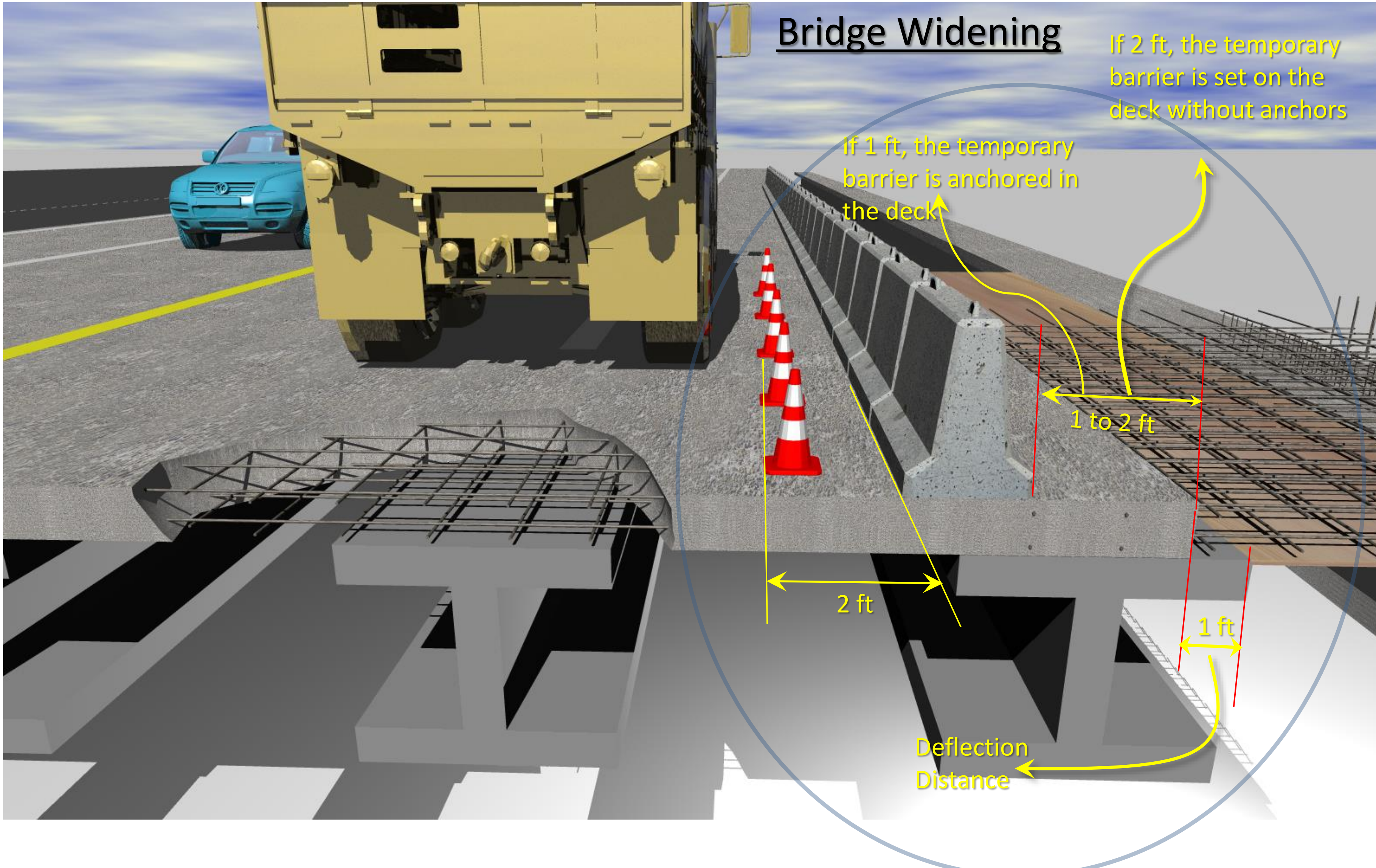
4.2.4 Deck Slab Design [9.7.2][9.7.3] (Rev. 01/10)

A. Empirical Design Method: The empirical design method per ***LRFD*** [9.7.2.4] is not permitted.

*Commentary: The empirical design method is not permitted because of the potential for future widening or phased construction and associated traffic control impacts in order to comply with **LRFD** [9.7.2.4].*


Bridge Widening
Extend the steel reinforcement

Potential for future widening or phased construction and associated traffic control impact



States that do not use the Empirical Deck Design Method

The main reasons for not using the empirical design method are:

1. Do not use the empirical deck design method for bridges constructed in stages or subject to future widening (Florida).

2. Empirical deck design method does not allow the use of precast prestressed deck panels (Missouri).

3. Preference for the traditional AASHTO design methodology or other standard design used by the state (Pennsylvania).

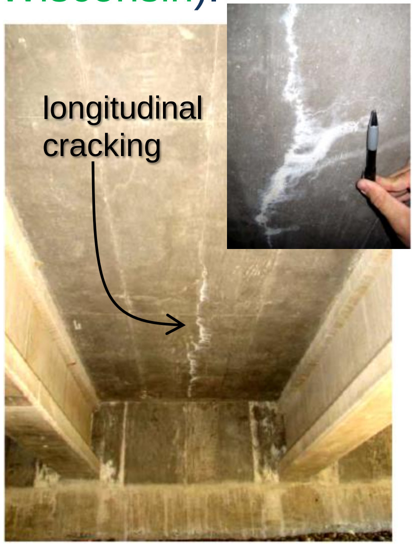
4. Larger girder spacing concerns (Tennessee).

5. Lack of experience and data regarding bridge life span (comfort level) (Wisconsin).

6. Concerns that a reduction in the deck reinforcing would result in a reduction of the service life of the deck (South Carolina).

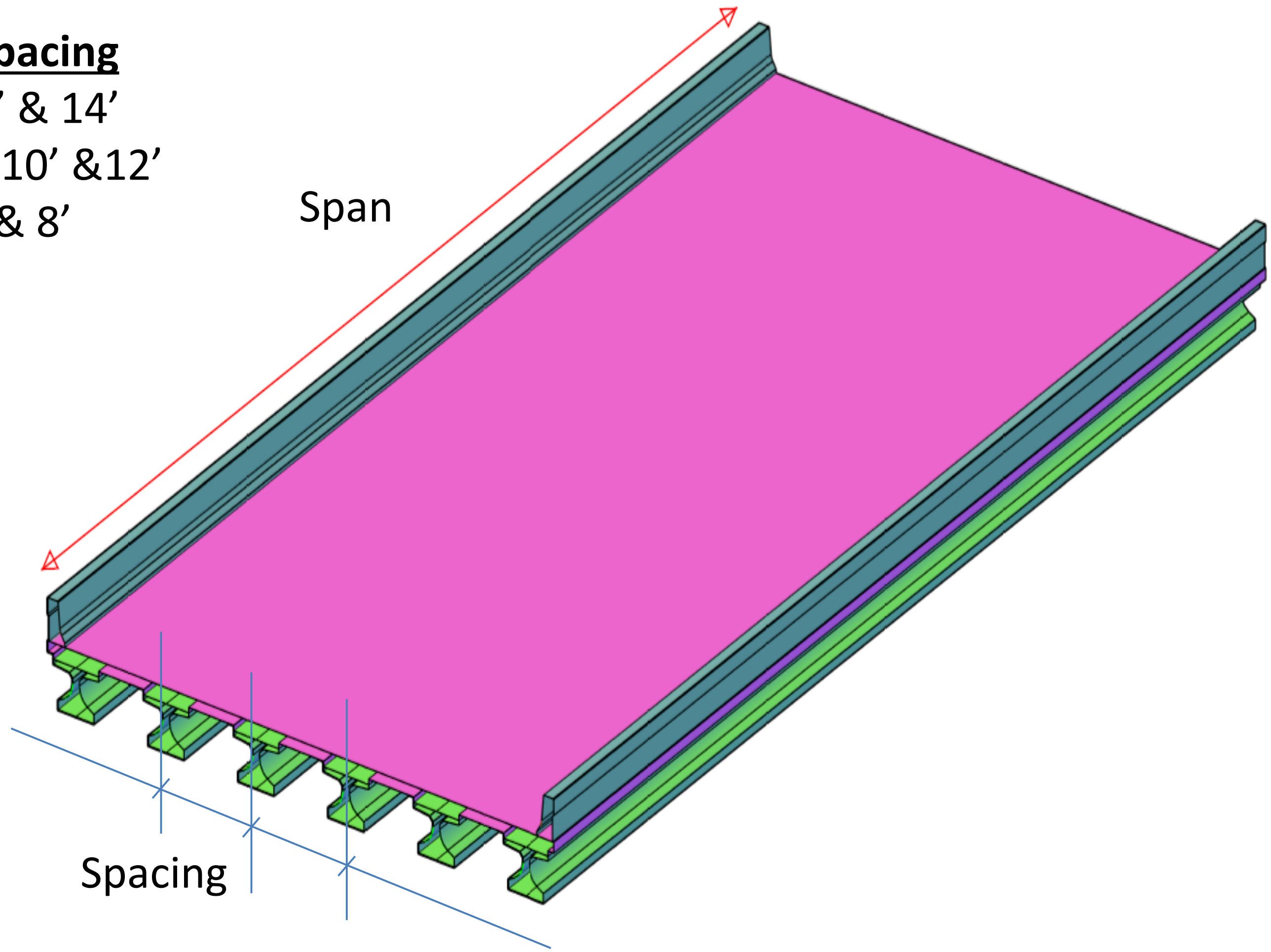
1. Experienced increased longitudinal cracking in the deck. Other crack patterns similar to traditionally designed bridges (Iowa).

1. Tried the empirical deck design method with a few bridges in the 1990's, did not like the results. Had issues with shrinkage cracking, no longer used (Oregon).

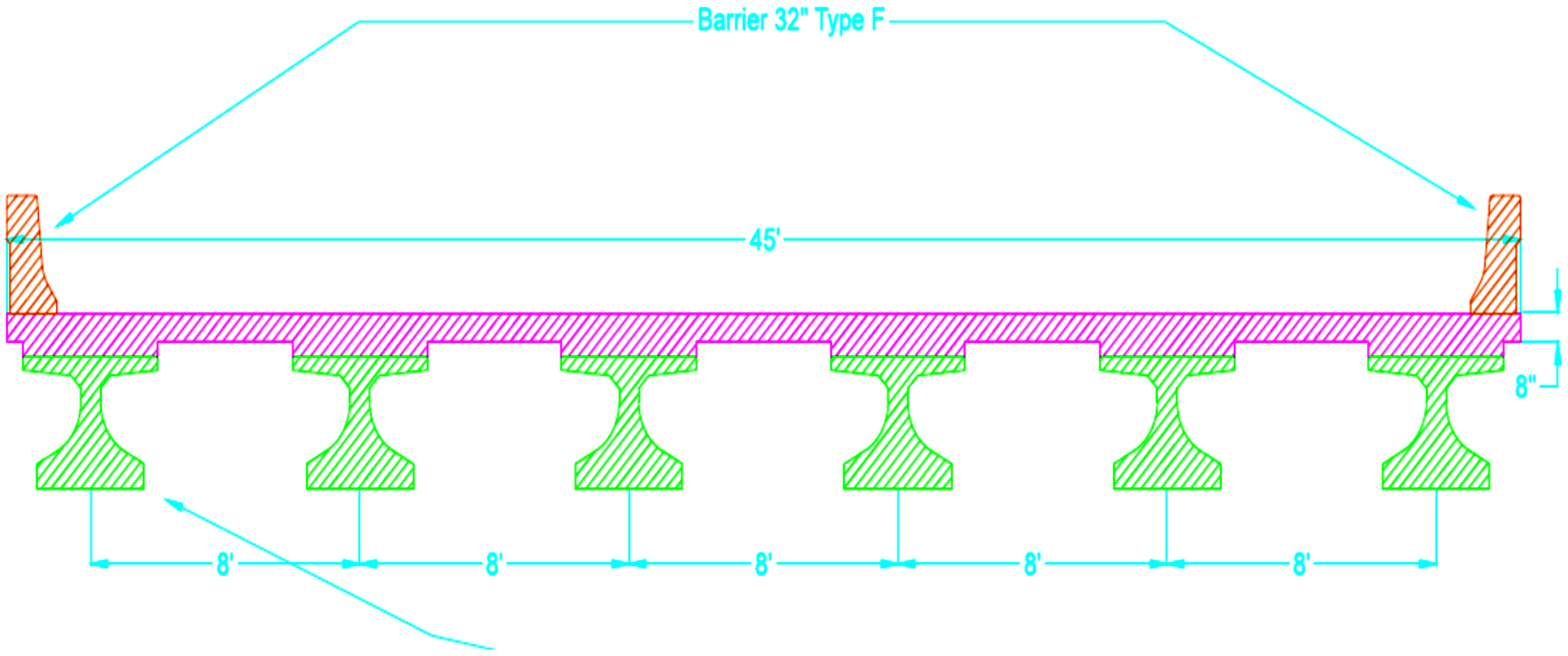


3-D Nonlinear FE Approach

Spans	**Spacing**
1- 80’	12’ & 14’
2- 90’	8’, 10’ &12’
3- 100’	6’ & 8’

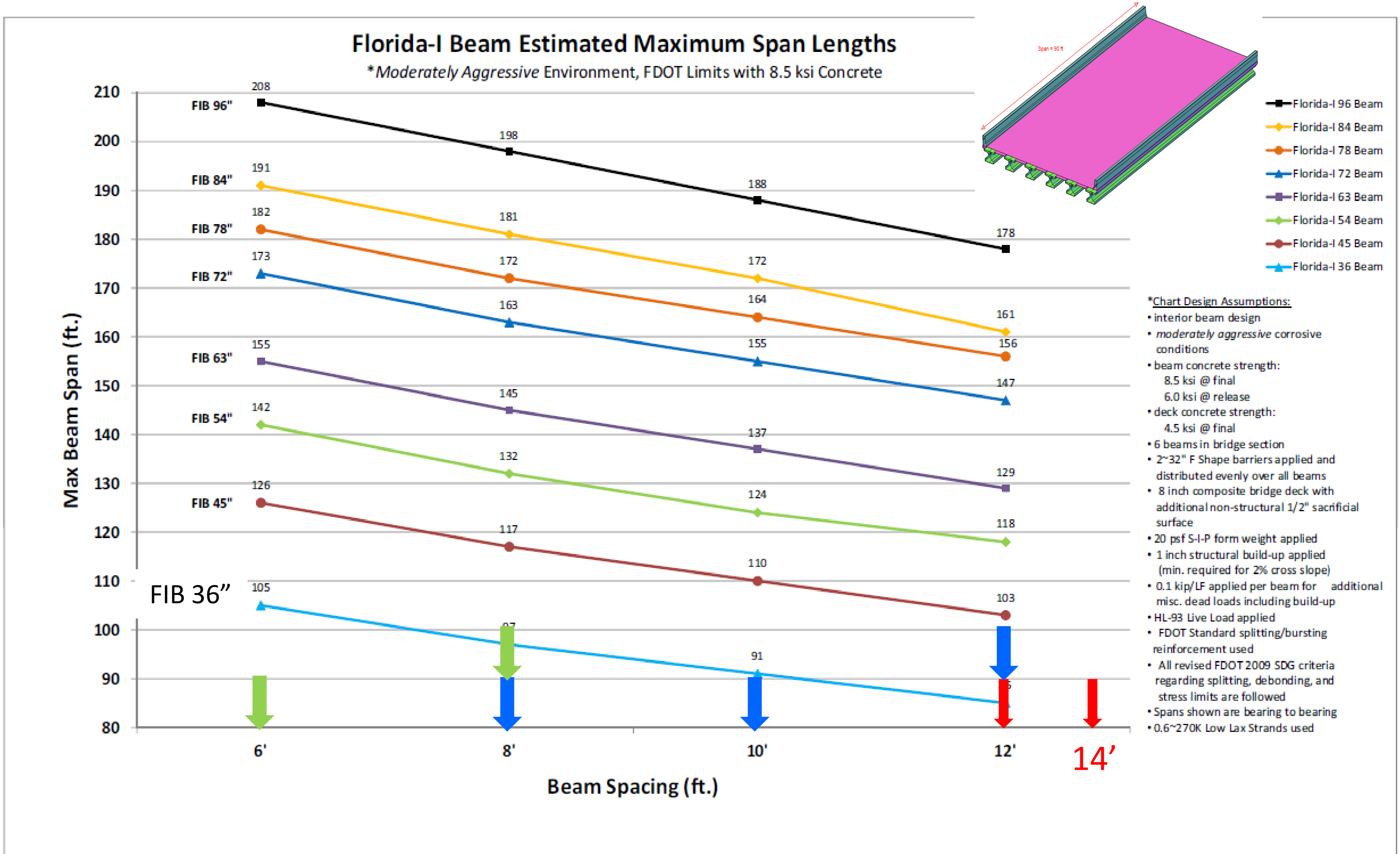


Proposed FE Approach

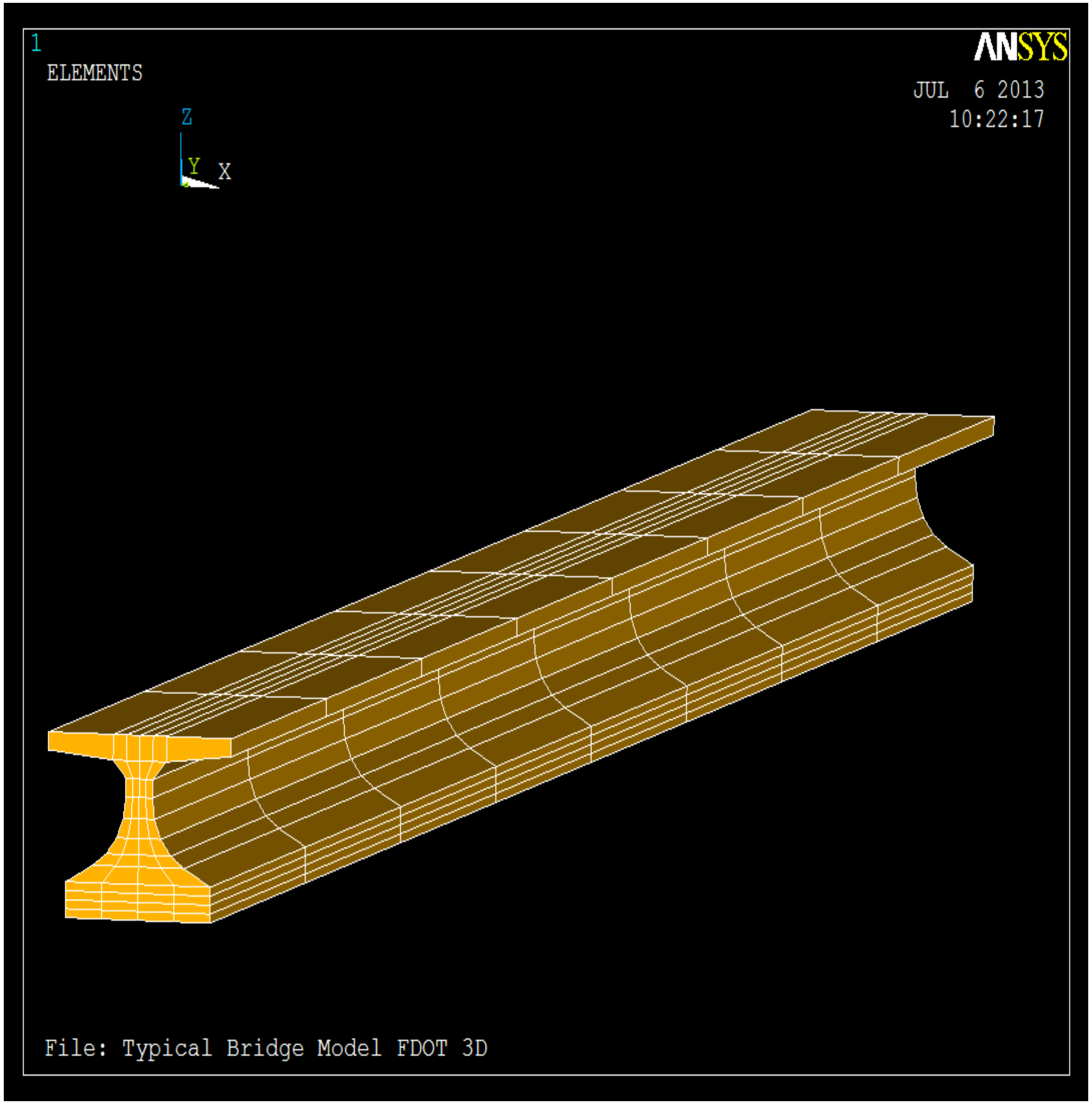


Design Aids

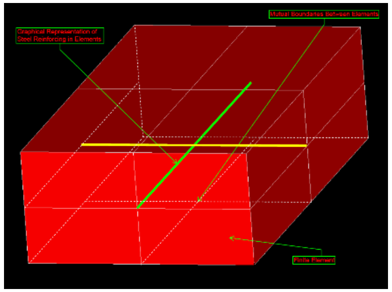
80'x12' & 80'x14'   90'x8' & 90'x10' & 90'x12'   90'x8' & 90'x10' & 90'x12'



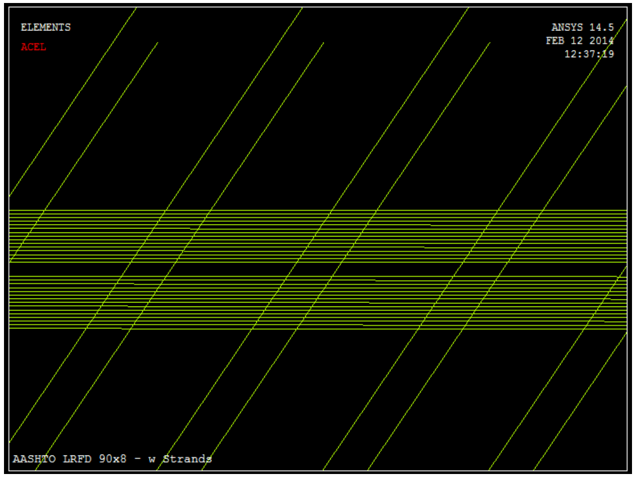
Steel Reinforcement is expressed in terms of
1- real constants (Using Smeared Model) and
2- links



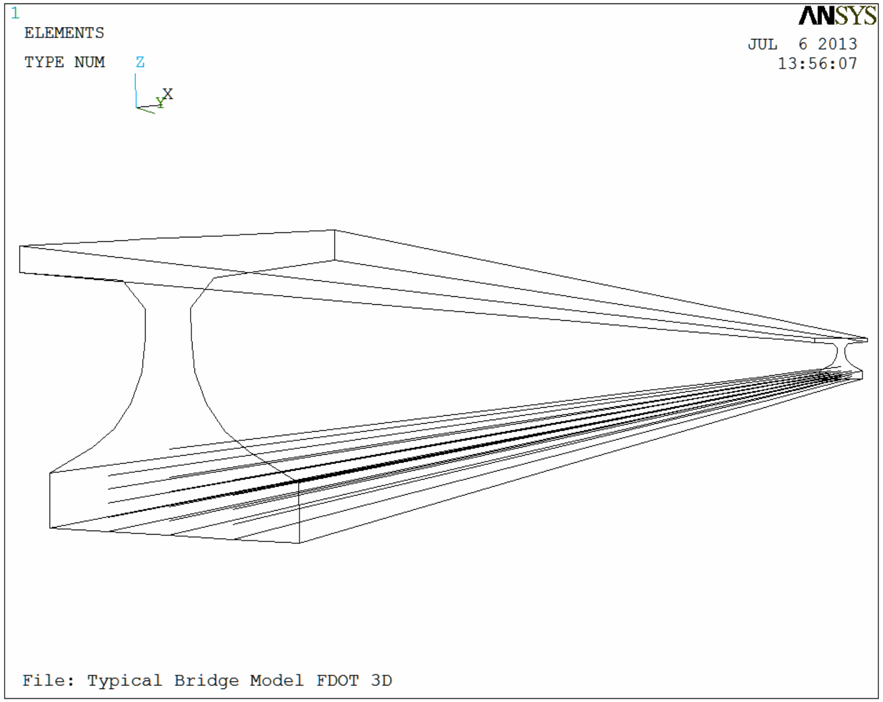
Smeared Model

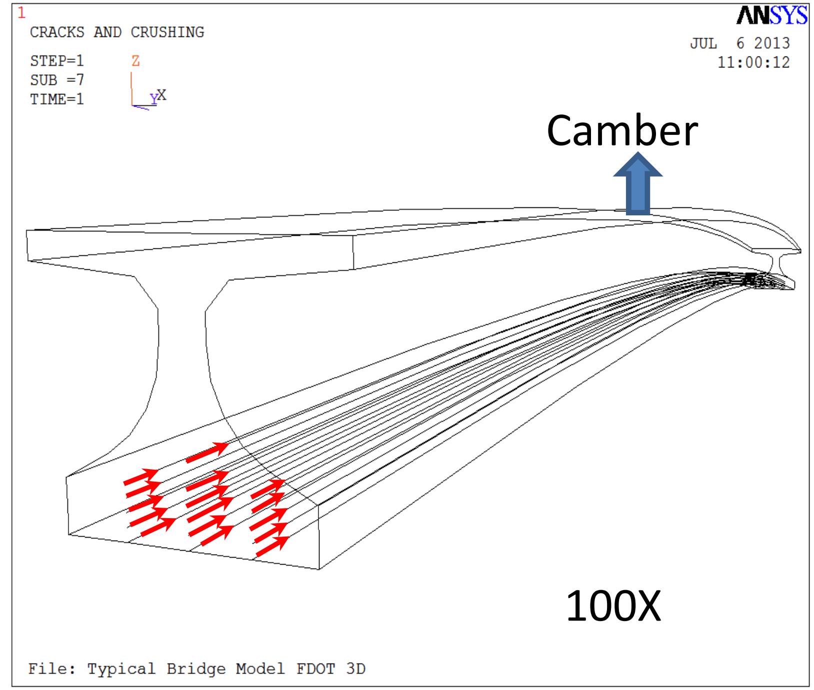


links



Pre-tension Stresses





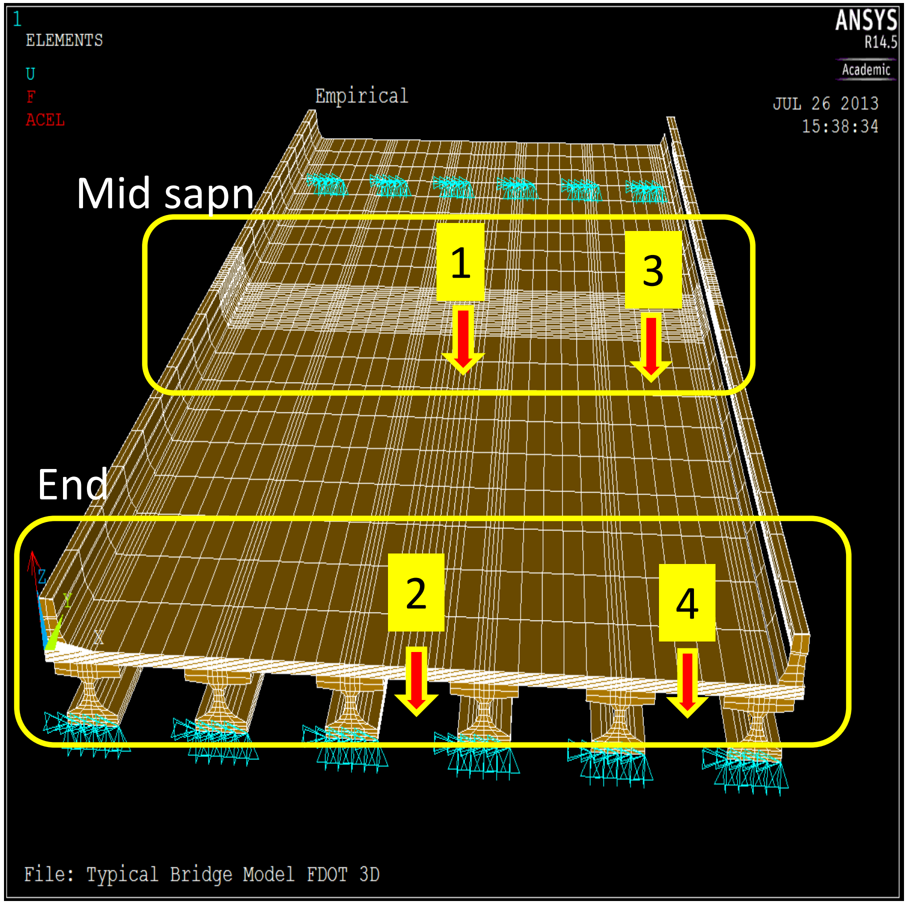
Instructions for Design Standards
Index 20010 Series Prestressed Florida-I Beams (Rev. 01/12)

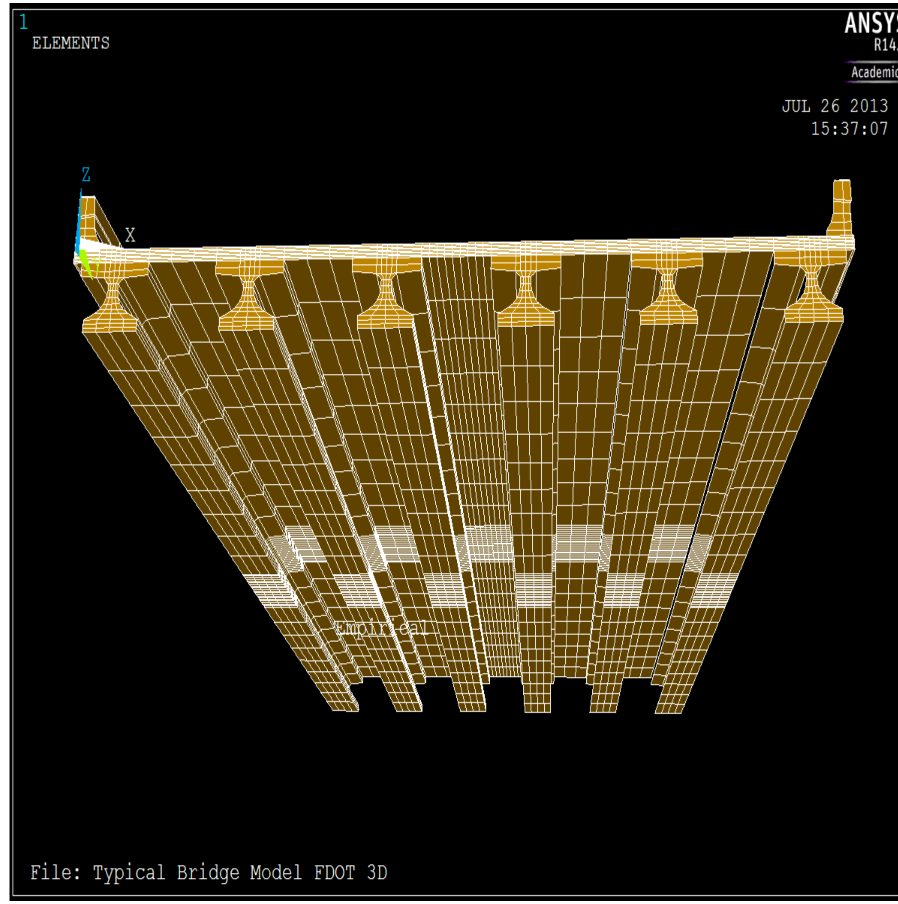
Topic No. 625-010-003-i
Fiscal Year 2012/2013

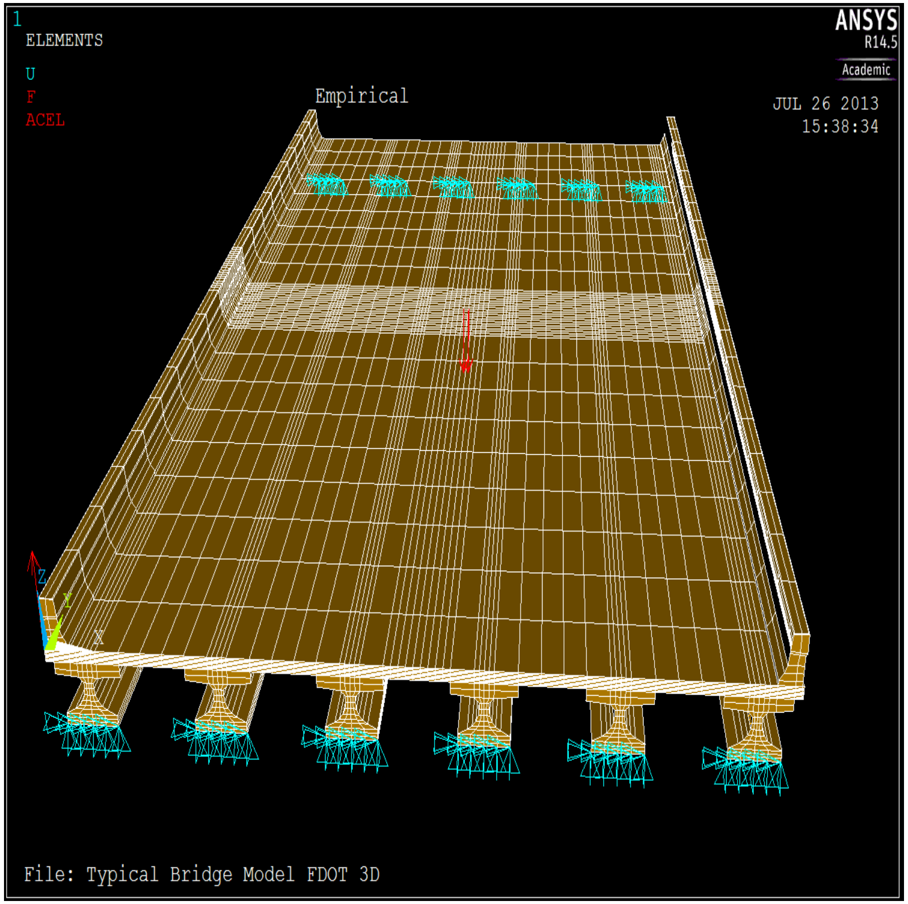
Index No.	Beam Type	Max. Bonded Prestress Force	Last Revision Date
20036	Florida-I 36	1450 Kips	07/01/09
20045	Florida-I 45	1670 Kips	07/01/09
20054	Florida-I 54	1740 Kips	07/01/09
20063	Florida-I 63	1740 Kips	07/01/09
20072	Florida-I 72	1980 Kips	07/01/09
20078	Florida-I 78	2230 Kips	07/01/09
20084	Florida-I 84	2375 Kips	07/01/10
20096	Florida-I 96	2375 Kips	07/01/10

Do not apply losses when calculating the Bonded Prestress Force.

Load Application



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ELEMENTS
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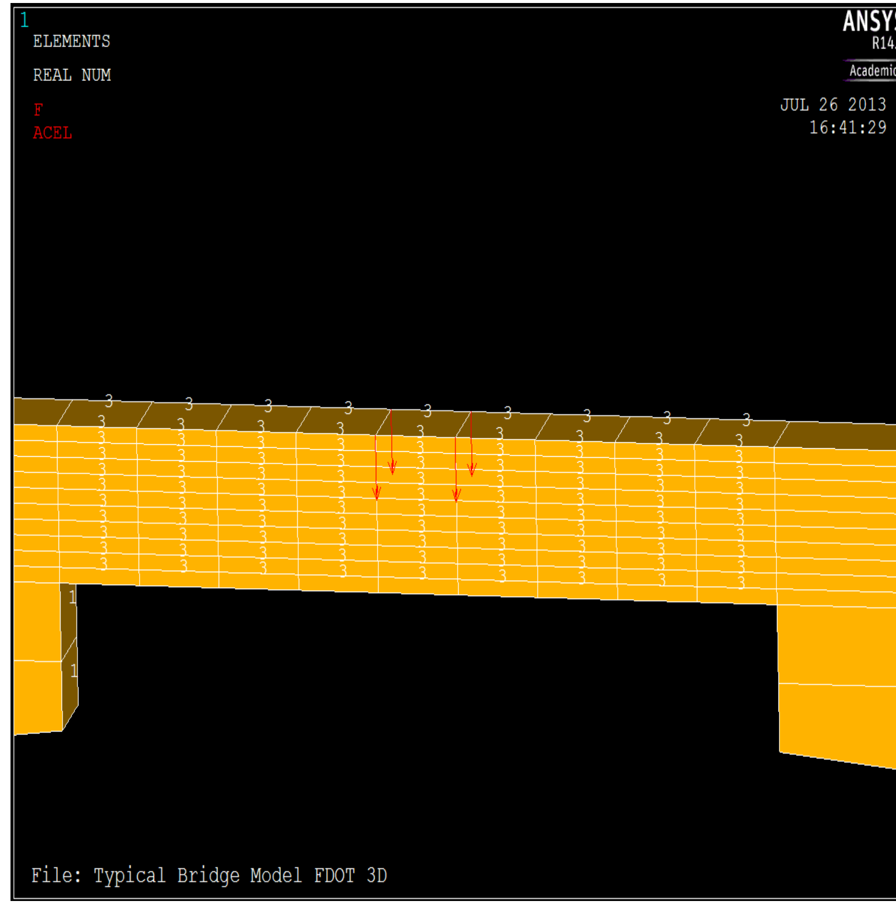
Y
X
File: Typical Bridge Model FDOT 3D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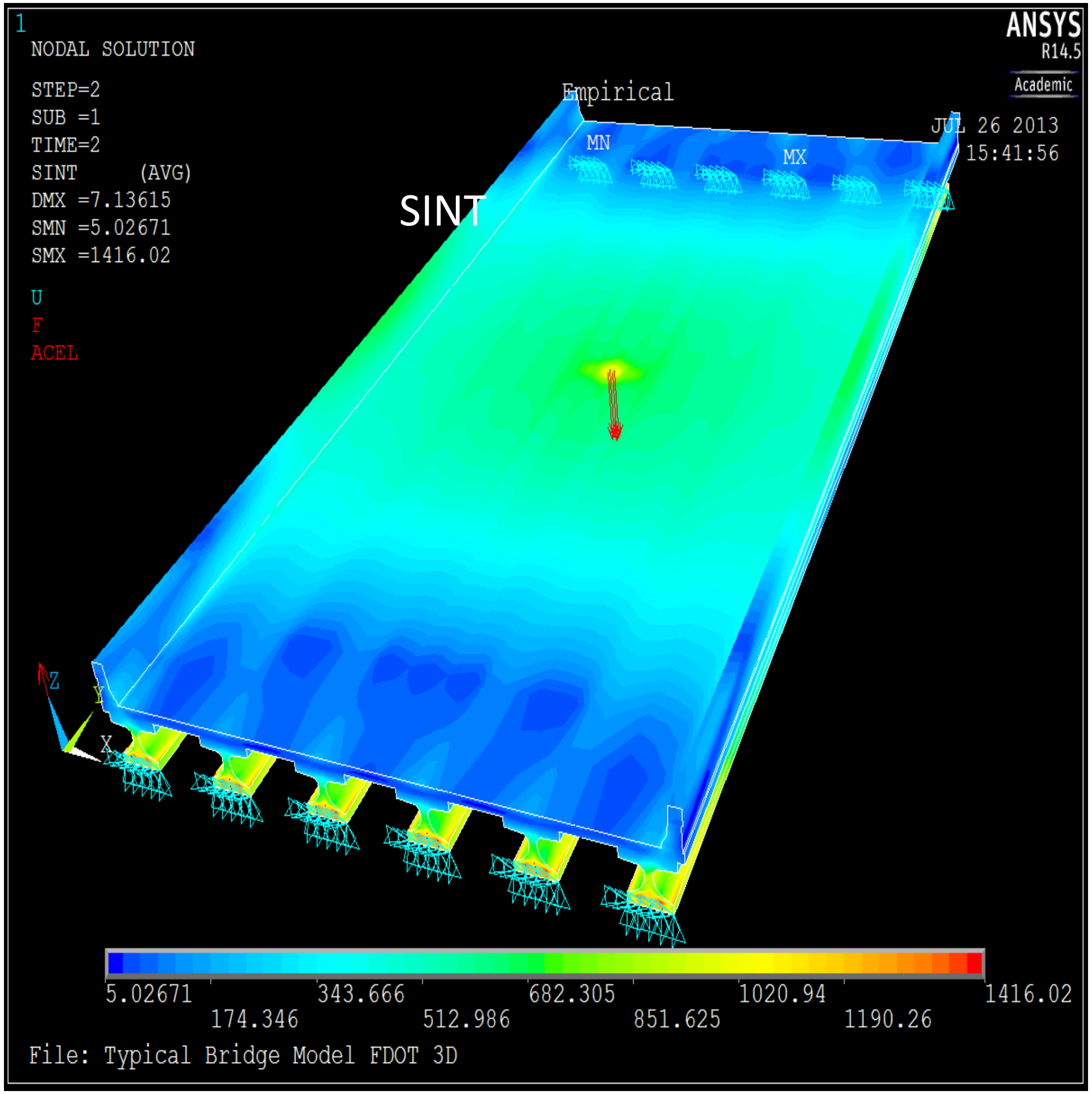
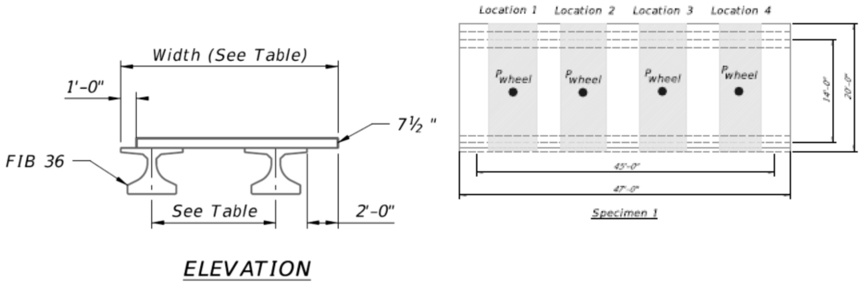
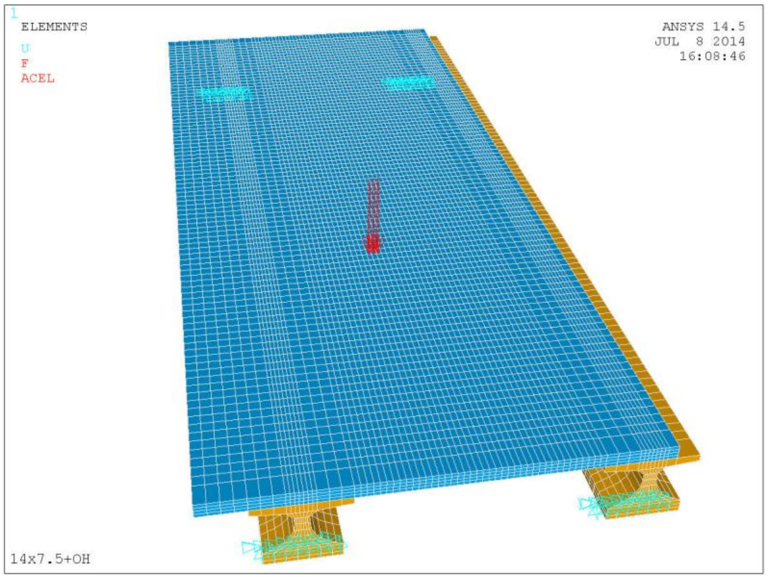


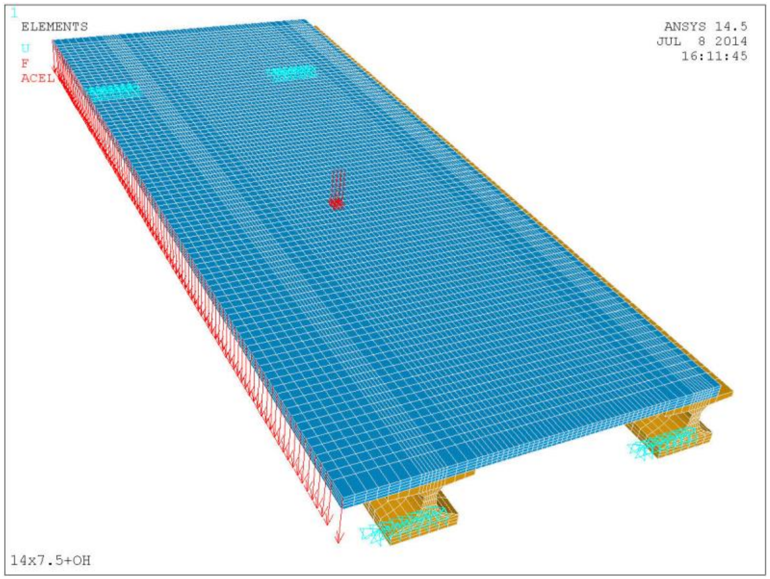
The differences in stresses between AASHTO LRFD and Empirical deck design methods are negligible.

Finite Element Model of the Lab Specimen



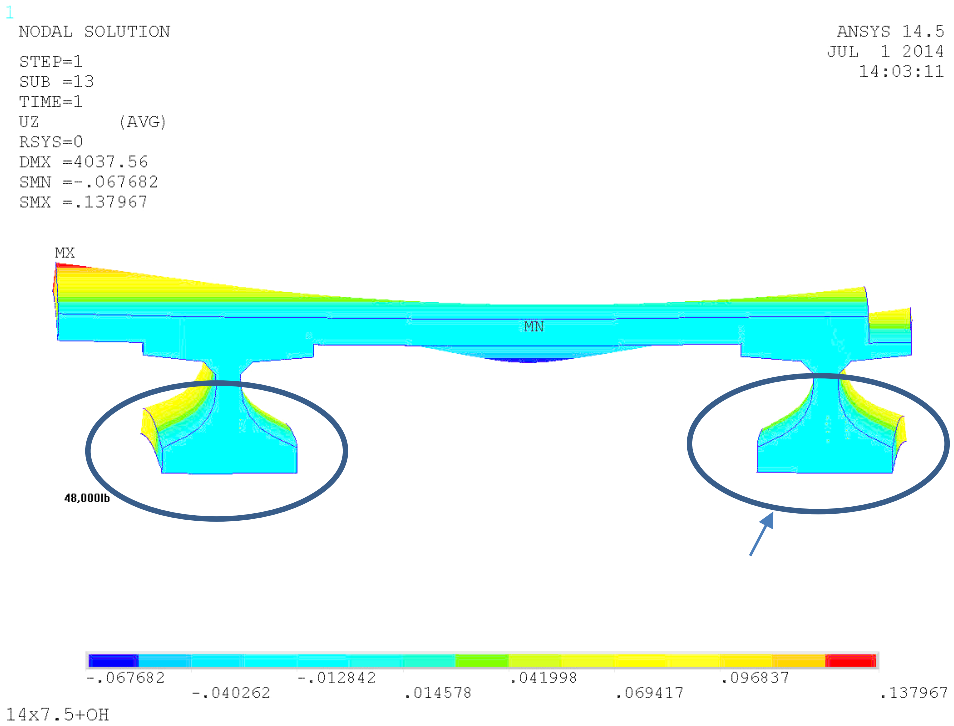


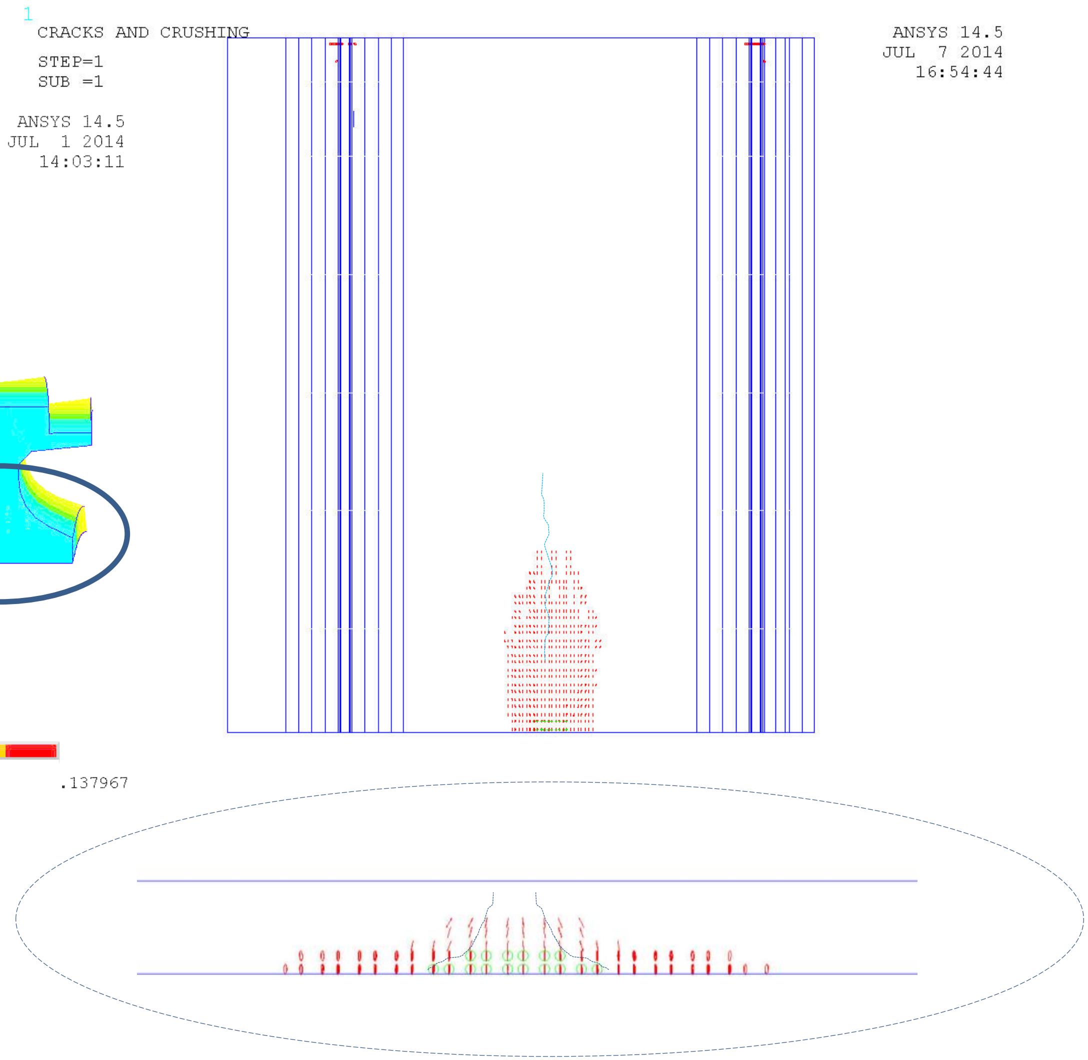
Finite Element Model of FIB 36 and Span 14 ft. With Point Load



Finite Element Model of FIB 36 Loaded at the Mid-span and the Edge

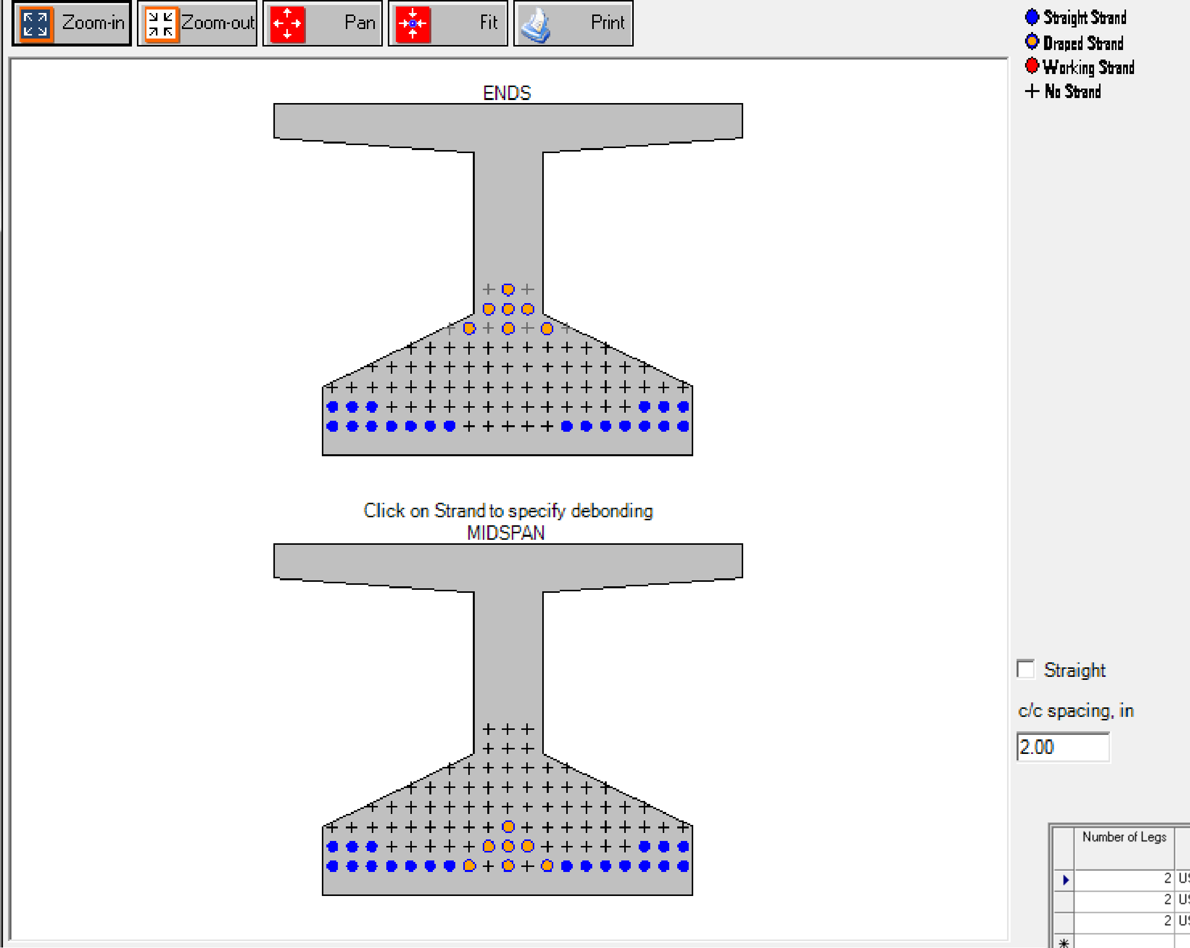
Finite Element Model of the Lab Specimen



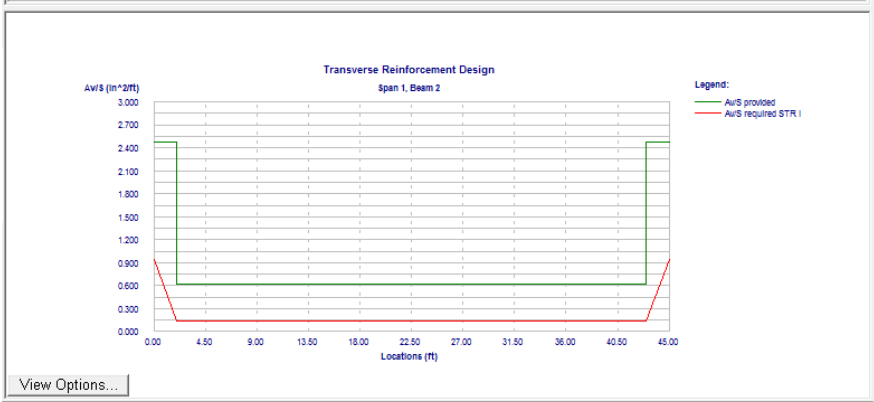


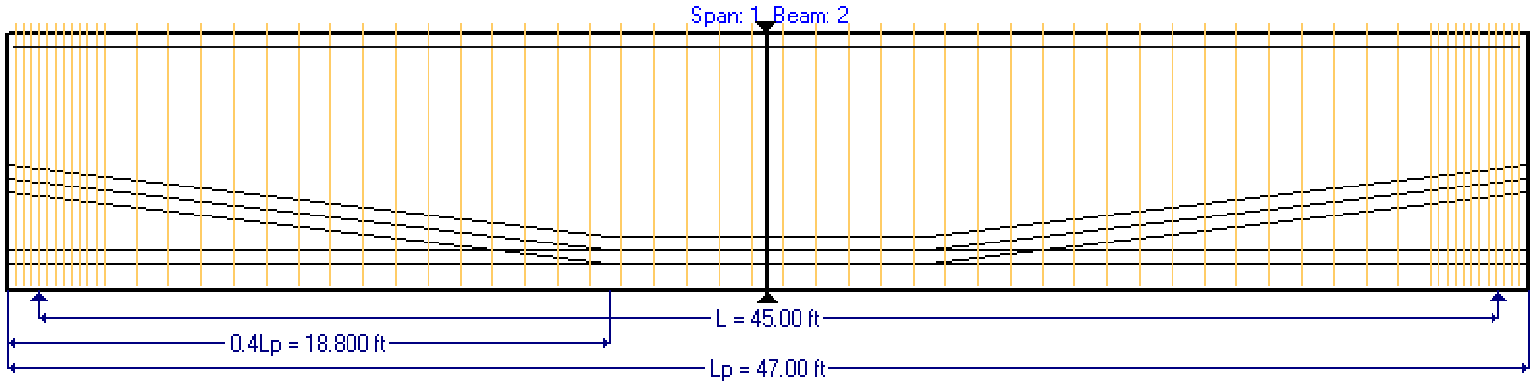
Crack Pattern under the point Load at the Mid-Span of the Slab

CONSPAN Design

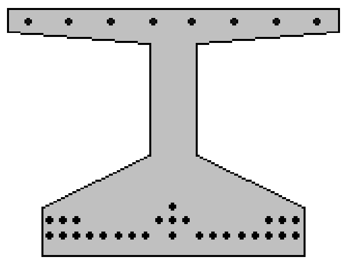


Number of Legs	Stirrup Size	Stirrup Area (in^2)	Stirrup Spacing (in)	Extends to Deck	Start (ft)	End (ft)
2	US#5[M16]	0.620	3.00	Yes	0.0000	3.0000
2	US#5[M16]	0.620	12.00	Yes	3.0000	44.0000
2	US#5[M16]	0.620	3.00	Yes	44.0000	47.0000





CROSS SECTION at Location: 0.5 L



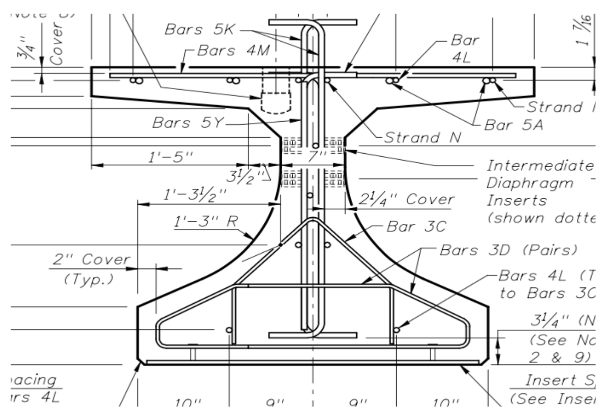
DESIGN SUMMARY

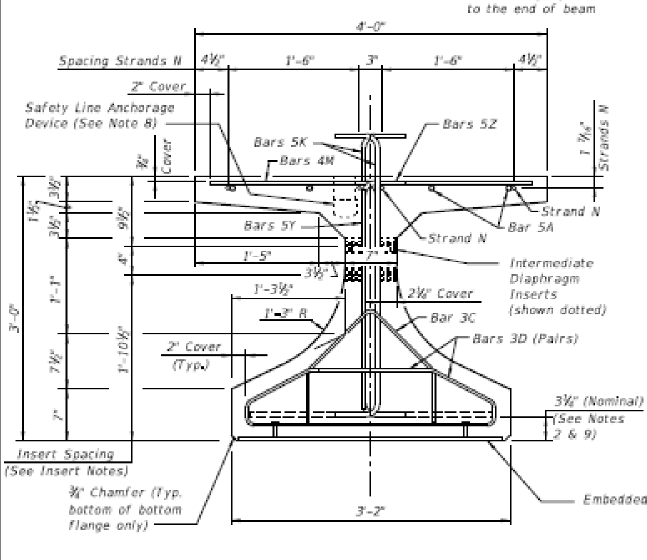
Straight Strands = 20
Draped Strands = 7

Ycg = 3.81 in
Pull = 1186.45 kips

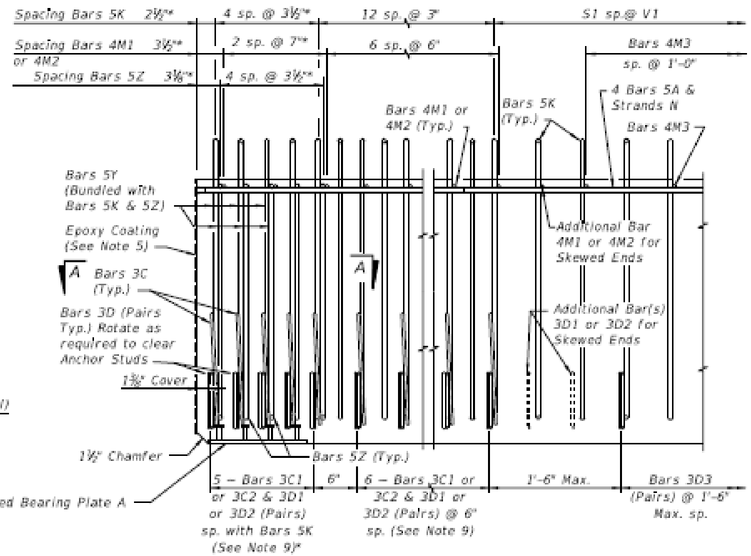
Per Location
Releases Stresses = OK
Final Stresses = OK
Ultimate Moment = OK
Debonded Strands = 0

Per Beam





END VIEW



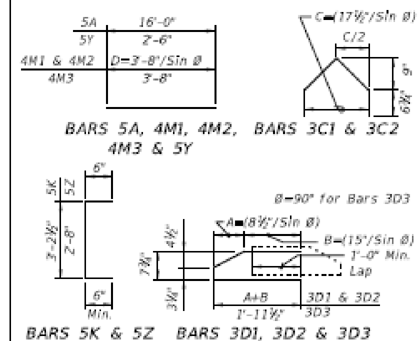
ELEVATION AT END OF BEAM
(Flanges Not Shown For Clarity)
(End 1 Shown, End 2 Similar)

CONVENTIONAL REINFORCING BAR BENDING DETAILS

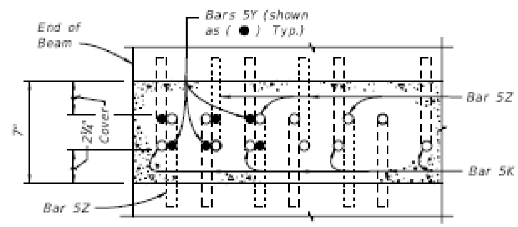
BILL OF REINFORCING STEEL

MARK	NOTE NUMBERS	SIZE	NUMBER REQUIRED	LENGTH (NOTE 1)
A	—	5	8	16'-0"
C1	9, 10 & 11	3	11 (End 1)	Varies
C2	9, 10 & 11	3	11 (End 2)	Varies
D1	9, 10, 11 & 14	3	22 (End 1)	Varies
D2	9, 10, 11 & 14	3	22 (End 2)	Varies
D3	9 & 14	3	See Table	4'-3"
K	2, 9, 11 & 13	5	See Table	4'-2"
M1	9 & 10	4	9 (End 1)	Varies
M2	9 & 10	4	9 (End 2)	Varies
M3	9	4	See Table	3'-8"
N	3 & 4	⅜" Ø Strand	4	Dim. L
Y	9 & 11	5	12	2'-6"
Z	2, 9, 11 & 13	5	10	3'-8"

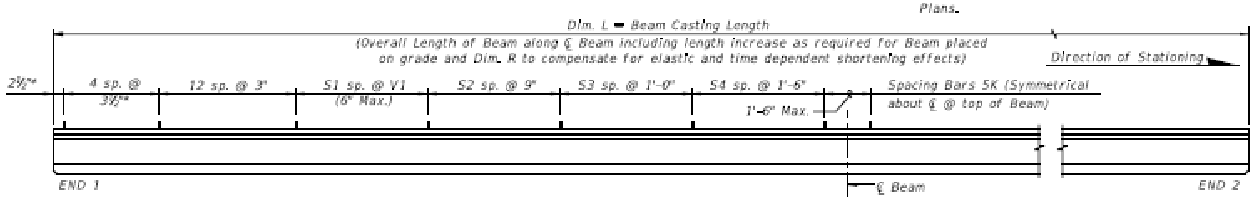
BENDING DIAGRAMS (See Note 1)



NOTES:
A. Work this Index with Index No. 20010 - Typical Florida-I Beam Details and Notes and the Florida-I Beam - Table of Beam Variables in Structures Plans.
B. For referenced notes, see Index No. 20010.
C. For Dimensions A, B, C, D, L, R & V1 and number of spaces S1 thru S4, see Florida-I Beam - Table of Beam Variables in Structures Plans.



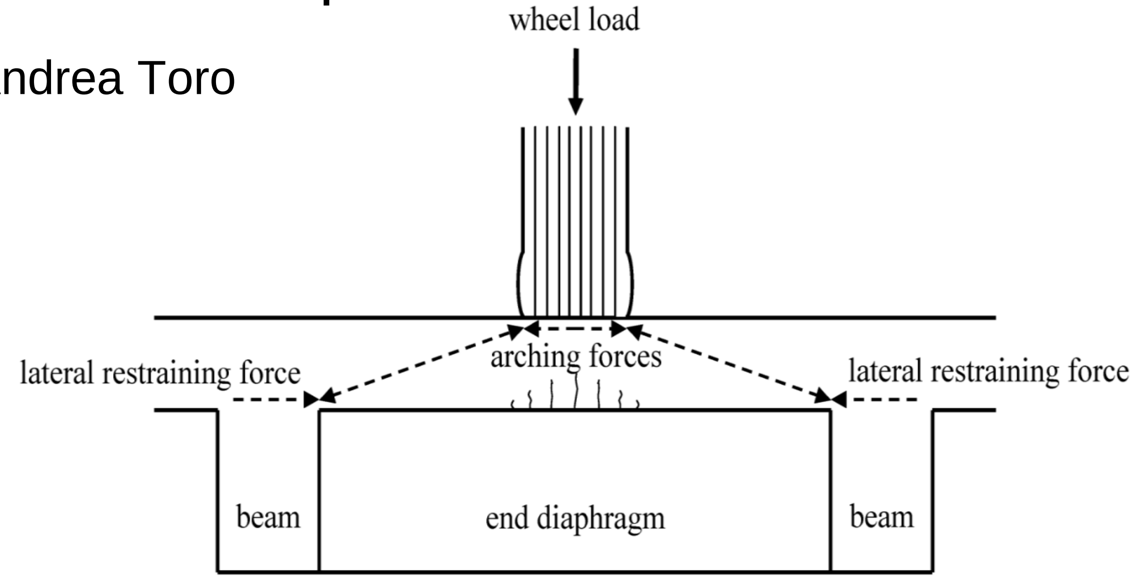
SECTION A-A FOR CONVENTIONAL REINFORCING
(Showing Bars 5K, 5Y & 5Z Only)



ELEVATION

Effect of lateral stiffness on bridge deck performance

By: Andrea Toro



Objectives:

- compressive membrane action (CMA)
- Compare the ultimate capacity predicted by four different methods.
- Determine the influence of lateral stiffness of support girders on the compressive membrane action, the behavior of deck slab, bridge deck ultimate capacity, and slab mode of failure.

methods of analysis

- British Standards (BS 5400)
- American Concrete Institute (ACI 318-05)
- UK Highway Agencies (BD81/02)
- Taylor, Rankin, and Clelands (TRC) approach

methods of analysis

British Standards (BS 5400)

- Bending capacity

$$M = A_s f_y d \left(1 - \frac{0.746 A_s f_y}{f_{cu} b d}\right)$$

- Flexural capacity under concentrated load $M = 0.08P\ kN \cdot m/m$

- Shear capacity under concentrated load

$$P_{vs} = 0.79 \cdot \sqrt[3]{100 \cdot \frac{A_s}{bd}} \cdot \sqrt[3]{\frac{fcu}{25}} \cdot \sqrt[4]{\frac{500}{d}} \cdot b_o \cdot d$$

American Concrete Institute (ACI 318-05)

- Bending ($M = \rho \cdot f_y \cdot d^2 \left(1 - \frac{0.5\rho f_y}{\beta \cdot f'c}\right)$

- Flexural capacity under concentrated load

$$M = 0.08P\ kN \cdot m/m$$

- Shear capacity under concentrated load

$$P_{vs} = 4 \cdot \sqrt{f'c} \cdot b_o d$$

methods of analysis

UK Highway Agencies (BD81/02)

- Elastic-plastic concrete stress block derived as

$$\varepsilon_c = (-400 + 60f'c - 0.33f'c^2) \times 10^{-6}$$

- McDowell's non-dimensional arching parameter

$$R = \frac{\varepsilon_c \cdot L_r{}^2}{h^2}$$

- Arching moment ratio

$$M_r = 4.3 - 16.1\sqrt{3.3\times10^{-4} + 0.1243R}$$

- McDowell's non-dimensional parameter for deflection

$$u = -0.15 + 0.36\sqrt{0.18 + 5.6R}$$

- Maximum arching moment coefficient

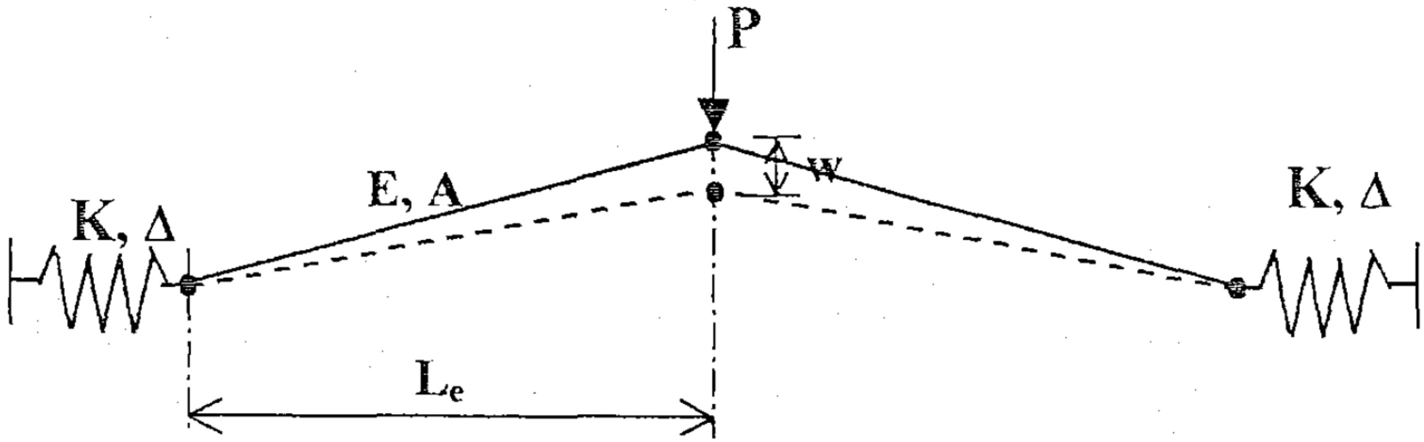
$$k = 0.0525\left(4.3 - 16.1\sqrt{3.3\times10^{-4} + 0.1243R}\right)$$

- Effective reinforcement ratio

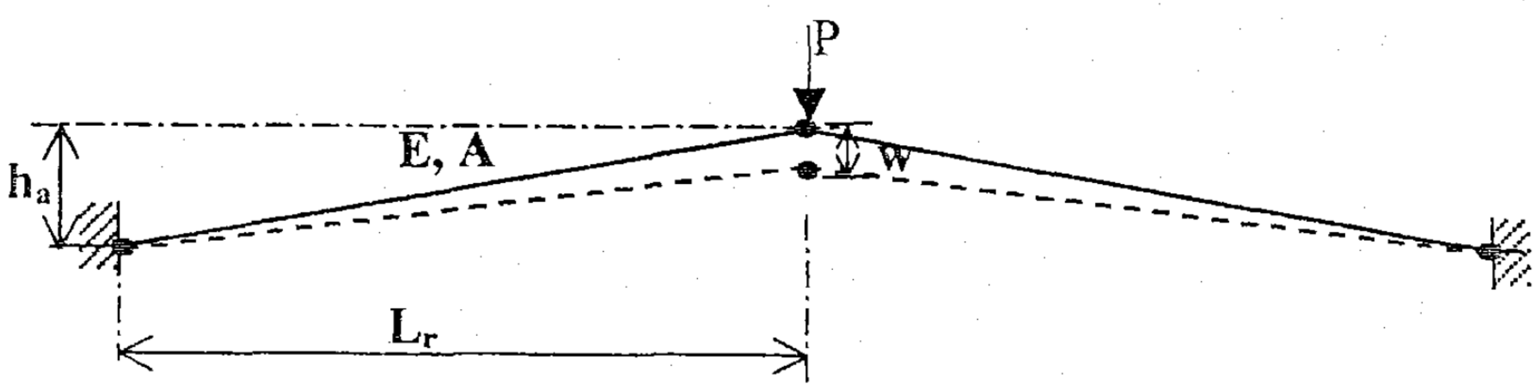
$$\rho_e = \frac{k \cdot f'c \cdot h^2}{240d^2}$$

- Shear punching stress

$$P_{pv} = 1.52 \cdot (c_x + d) \cdot d \cdot \sqrt{f'c} \cdot (100\rho_e)^{0.25}$$



a) elastically restrained arch



b) equivalent rigidly restrained arch

methods of analysis

Taylor, Rankin, and Cleland approach (TRC)

1. Effective width of loaded slab

$$L_e = \frac{L}{2} - \frac{c_x}{2}$$

2. Stiffness

$$b_{eff} = c_y + 2 \cdot L_e + 2h$$

$$E_c = 4.23\sqrt{f'c}$$

$$K_s = \frac{E_c h b_{beff}}{L_e}$$

$$A_b = \frac{\zeta L_e I_{yb}}{{b_{eff}}^3}$$

$$K_b = \frac{A_b E_C}{L_e}$$

$$K_d = \frac{\Sigma A_d E_c}{L_e}$$

$$K_r = \frac{1}{\frac{1}{K_b} + \frac{1}{K_d}}$$

3. Bending capacity

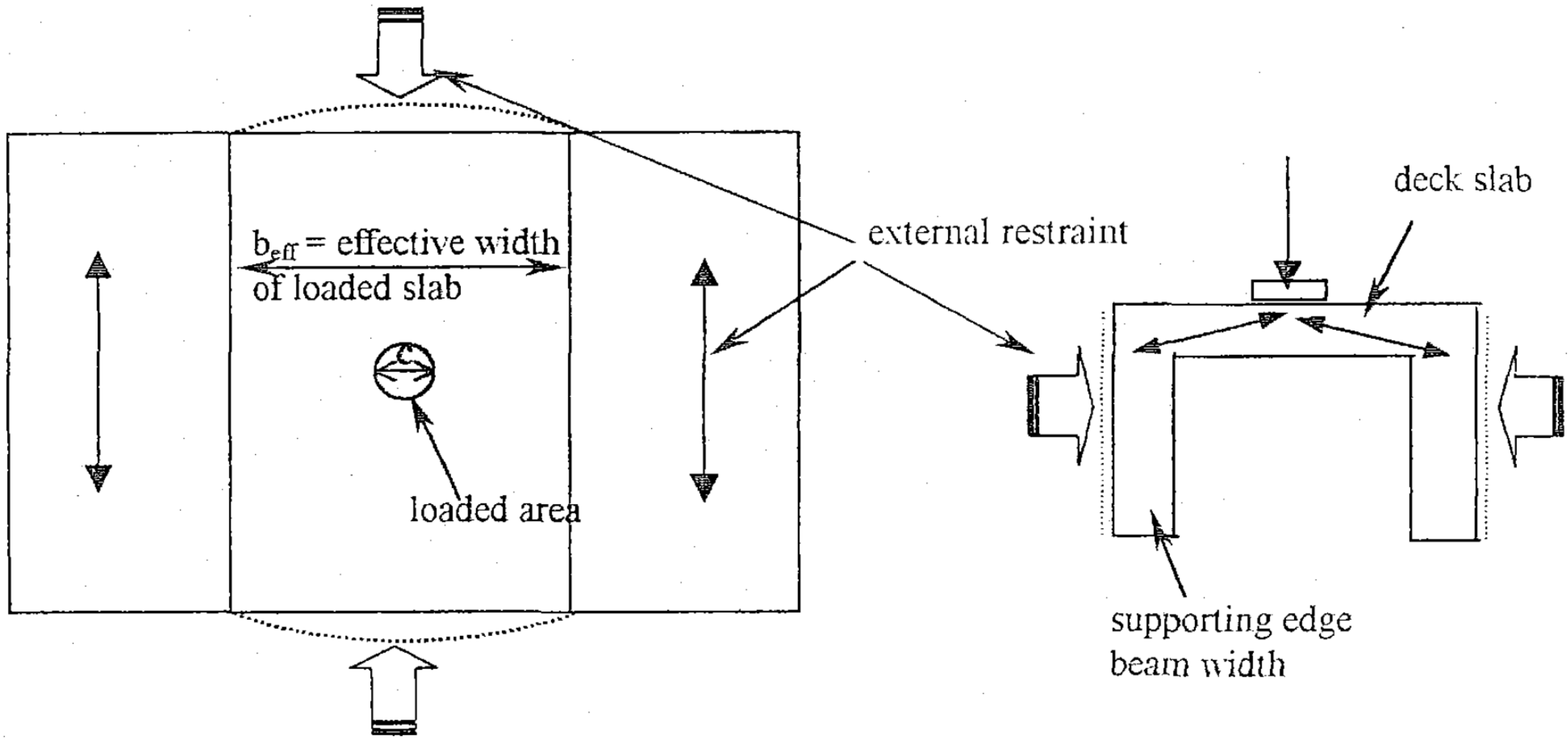
Depth of stress block, $\beta = 1 - 0.003\, f'c$ but <0.9

Depth of neutral axis, $x = \frac{f_y A_s}{0.67\, f'c\beta\, b}$

Lever arm, z=d – 0.5 β x

$$M_b = f_y A_s\, z$$

$$P_b = k_b M_b$$



b_{eff} = effective width
of loaded slab
loaded area
external restraint
deck slab
supporting edge
beam width

Numerical methods of analysis

Taylor, Rankin, and Cleland approach (TRC)

The length of the equivalent rigidly restrained slab strip and the contact area are dependent upon the degree of lateral restraint, therefore this required an iterative process between step 4 and 8 until the proportion of half the arching depth in contact with the support was constant.

4. Arching section

$$2d_1 = h - 2x\beta$$

new d_1 from previous iterations

5. Equivalent Rigidly Restrained Slab

$$A = \alpha b d_1$$

6. Arching p

$$L_r = L_e \sqrt[3]{\left(\frac{EA}{KL_e} + 1\right)}$$

$$\varepsilon_u = 0.0043 - [(f'c - 60)2.5\times10^{-5}] \quad \text{but} < 0.0043$$

$$\text{and } R = \frac{\varepsilon_u L_r^2}{4d_1^2}$$

$$\varepsilon_c = 2\varepsilon_u(1-\beta)$$

7. Deformation

R>0.26 ➔ u=0.31

0<R<0.26 ➔ $u = -0.15 + 0.36\sqrt{0.18 + 5.6R}$

8. Contact depth

$$\alpha = 1 - \frac{u}{2}$$

αd_1 use for refined arching action section above until value remains constant.

9. Arching capacity

R>0.26 ➔ $M_r = \frac{0.3615}{R}$

0<R<0.26 ➔ $M_r = 4.3 - 16.1\sqrt{3.3\times10^{-4} + 0.1243R}$

$$M_a = 0.168bf'cd_1^2 M_r \left({}^{L_e}/{}_{L_r}\right)$$

Numerical methods of analysis

Taylor, Rankin, and Cleland approach (TRC)

9. Arching capacity

$$P_a = k_a M_a$$

10. Flexural punching capacity

$$P_{pf} = P_a + P_b$$

11. Shear punching capacity

$$\rho_e = (\rho_e + \rho)\left(\frac{f_y}{320}\right) = \left(\frac{M_a + M_b}{M_b}\right)\left(\frac{f_y}{320}\right)\rho$$

$$P_{pv} = \frac{0.43}{r_f}\sqrt{f'c}(critical\ permeter)d\,(100\rho_e)^{0.25}$$

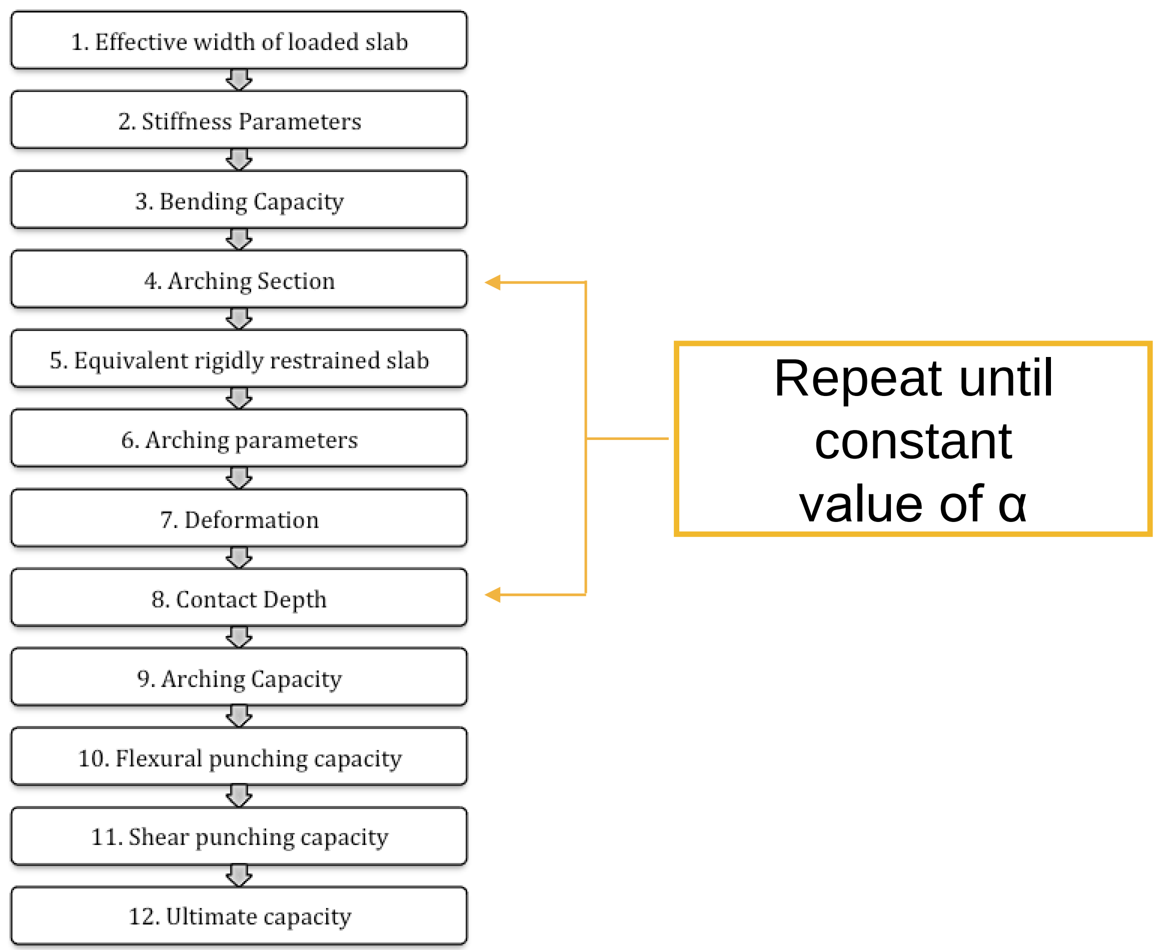
12. Ultimate Capacity

$$\text{If } P_{pf} < P_{pv} \rightarrow \quad P_p = P_{pf}$$

$$\text{If } P_{pf} > P_{pv} \rightarrow \quad P_p = P_{pv}$$

methods of analysis

Taylor, Rankin, and Cleland's approach (TRC)



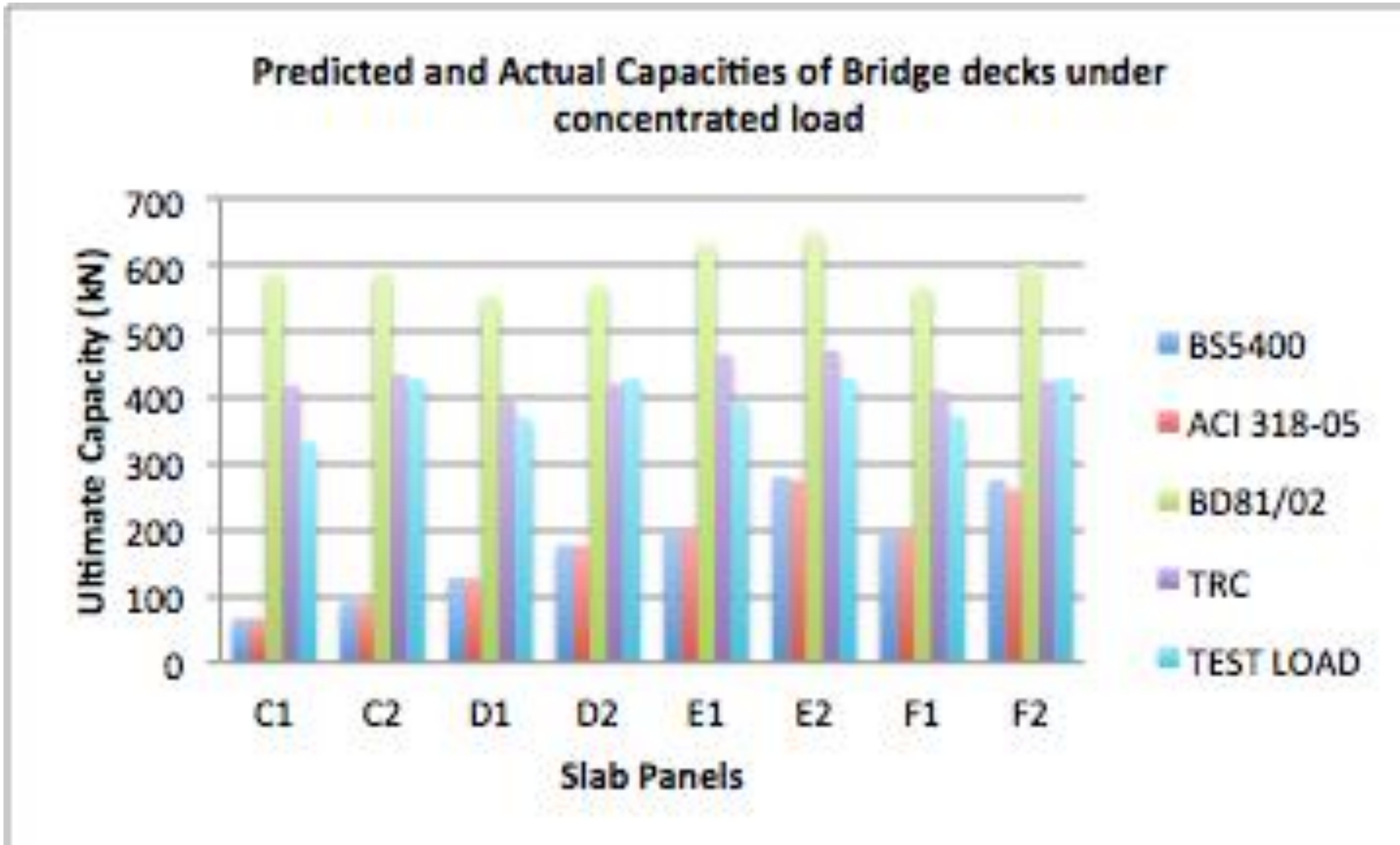
actual testing & MODEL Results

“Serviceability of bridge deck slabs with arching action”

by Taylor, S.E., Rankin, B., Cleland, B.J., and Kirkpatrick, J. (2007)

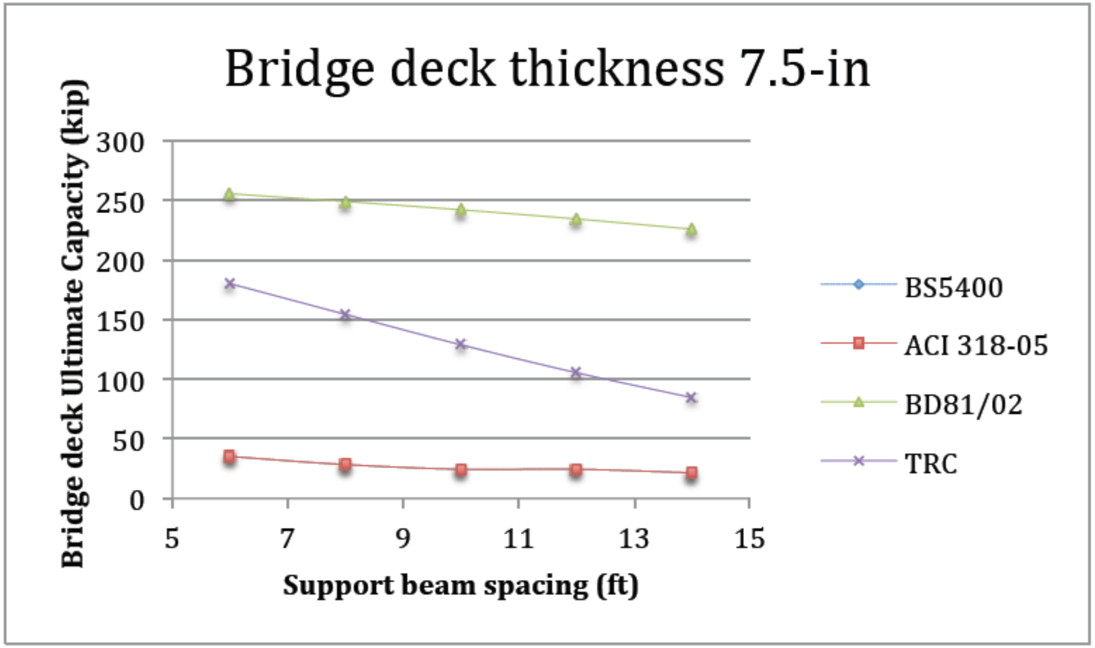
Capacity	BS 54000	ACI 318-05	BD81/02	TRC approach
Flexural	66.6 kN	66.2kN	-	504.422 kN
Shear	167.3 kN	214.1 kN	588.0 kN	418.829 kN
Ultimate	66.6 kN	66.2kN	588.0 kN	418.829 kN

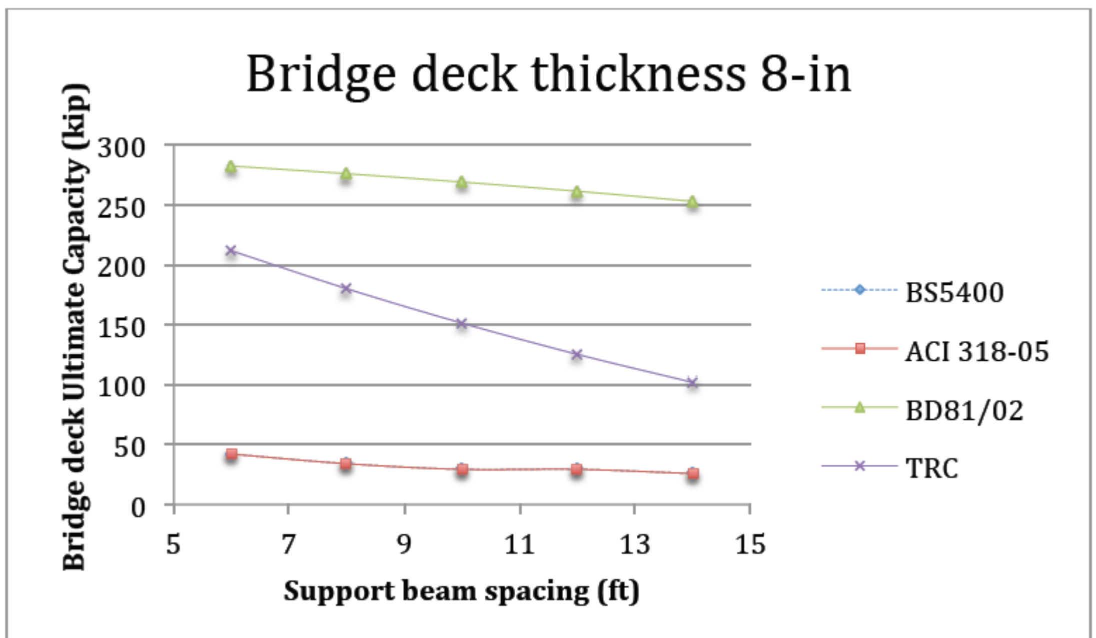
The actual test results showed that the panel’s maximum test load capacity was 333 kN


Predicted and Actual Capacities of Bridge decks under concentrated load
Ultimate Capacity (kN)
700
600
500
400
300
200
100
0
C1
C2
D1
D2
E1
E2
F1
F2
Slab Panels
BS5400
ACI 318-05
BD81/02
TRC
TEST LOAD

PARAMETERS FOR ANALYSIS

- 5 - different deck slab thickness (7.5, 8, 8.5, 9, 9.5 inches)
- 5 - different support beam spacing (6, 8, 10, 12, 14 feet)
- Steel reinforcement ratio 0.454%
- 80-foot span
- FIB-36 girder


Bridge deck thickness 7.5-in
Bridge deck Ultimate Capacity (kip)
300
250
200
150
100
50
0
5
7
9
11
13
15
Support beam spacing (ft)
BS5400
ACI 318-05
BD81/02
TRC


Bridge deck thickness 8-in
Bridge deck Ultimate Capacity (kip)
300
250
200
150
100
50
0
5
7
9
11
13
15
Support beam spacing (ft)
BS5400
ACI 318-05
BD81/02
TRC

Lateral Restraint analysis parameters (further analysis was performed based on the TRC approach, since it has resulted in a very significant contribution having a load and a carrying capacity close to the actual testing load)

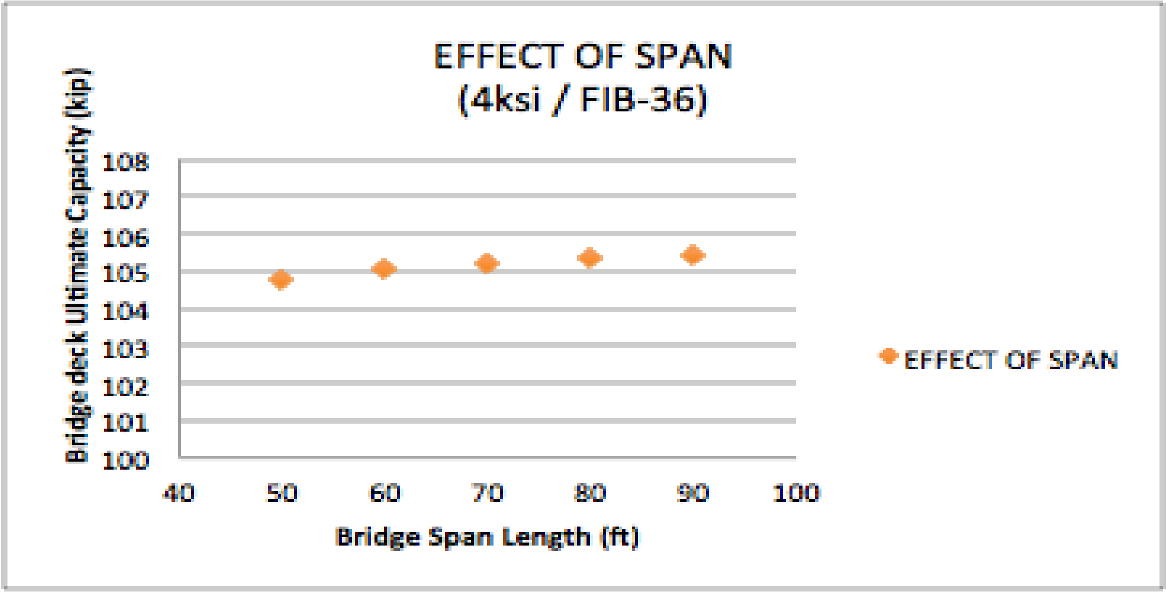
- 5 - different deck slab thickness (7.5, 8, 8.5, 9, 9.5 inches)
- 5 - different support beam spacing (6, 8, 10, 12, 14 feet)
- 5 - different bridge span lengths (50, 60, 70, 80, 90 feet)
- 4 - different types of girders (FIB-36, AASHTO type III, W44x335, built-up steel girder)
- 2- Steel reinforcement ratio (0.454%, 0.63%)
- 2 - different compressive concrete strengths (4, 5 ksi)

Support BEAM properties

	FIB-36	AASHTO TYPE III	W44X335	BUILT UP
CROSS SECTION AREA (in2)	806.58	560	98.5	106
Ix (in4)	127,564	125,390.35	31,100	99,734
Iy (in4)	81,131	12,216.56	1,200	2,884.55
Material	concrete	concrete	steel	steel
Modulus of Elasticity N/mm2 (ksi)	2.85E+04 (4.134E+03 ksi)	2.85E+04 (4.134E+03 ksi)	2.00E+05 (2.90E+04 ksi)	2.00E+05 (2.90E+04 ksi)
Rectangular load patch (in)	10x20	10x20	10x20	10x20

Effect of Bridge beam span

f'c=4ksi	FIB-36						
EFFECT OF SPAN				Flexural Arching punching capacity	Shear punching Capacity	Ultimate capacity	Type of failure
Length	Spacing	Thickness	ρ empirical	kip	kip	kip	
50	12'	7.5	0.45	104.802	170.495	104.802	Flexural
60				105.057	170.638	105.057	Flexural
70				105.225	170.733	105.225	Flexural
80				105.344	170.799	105.344	Flexural
90				105.433	170.849	105.433	Flexural

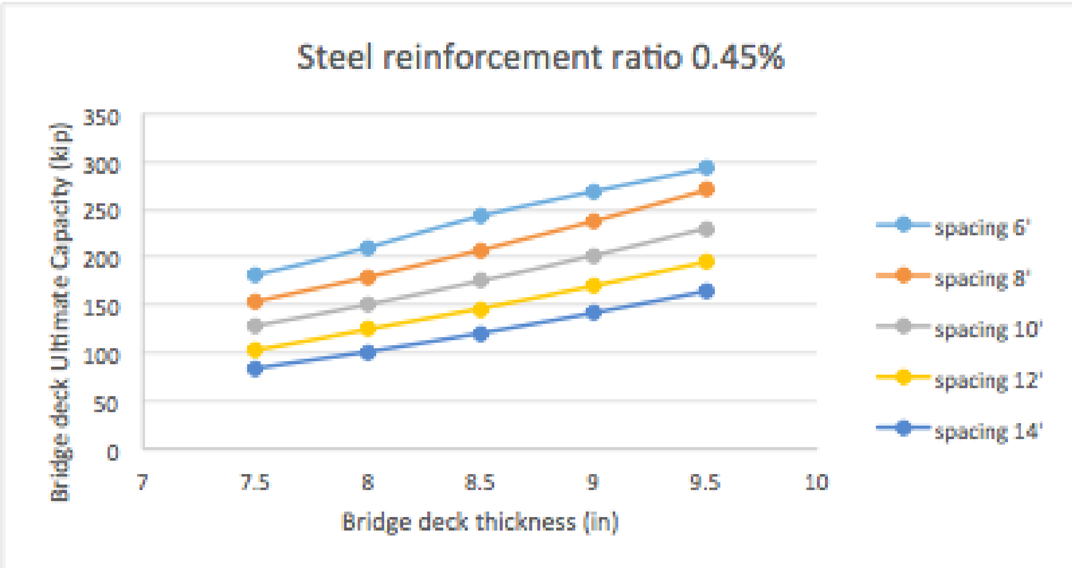


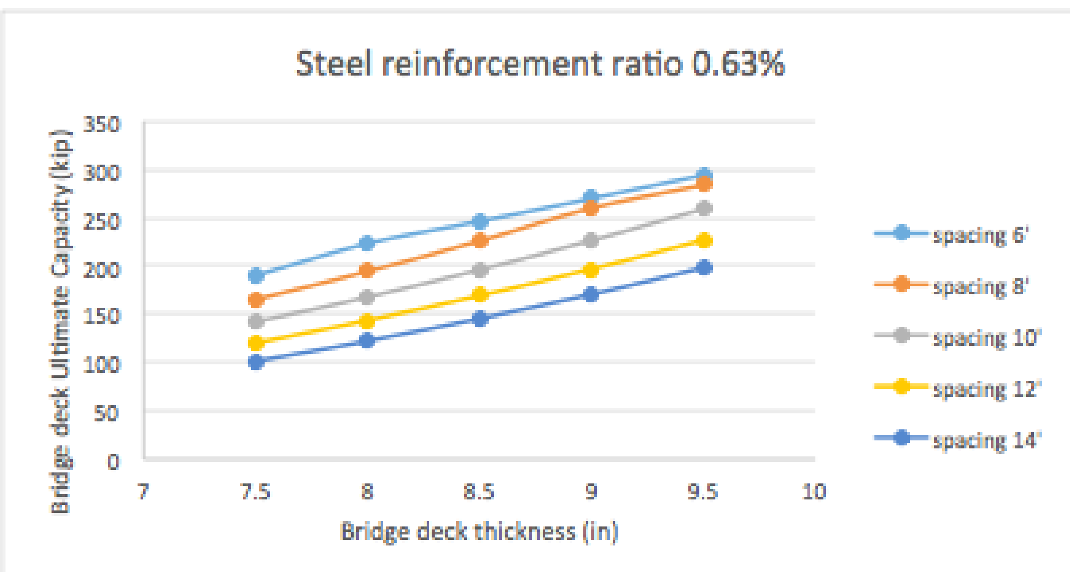
EFFECT OF COMPRESSIVE CONCRETE STRENGTH (f'c)

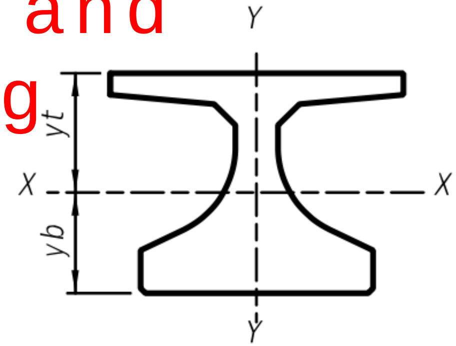
Effect of f'c					Flexural Arching punching capacity	Shear punching Capacity	Ultimate capacity	Type of failure
f'c (ksi)	Length (ft)	Spacing (ft)	Thickness (in)	ρ empirical (%)	kip	kip	kip	
4	80	12'	7.5	0.45	92.898	146.676	92.898	Flexural
5					105.344	170.799	105.344	Flexural
8					119.593	224.327	119.593	Flexural

effect of slab thickness and support beam spacing

(ρ=0.45% and 0.63%) - FIB-36

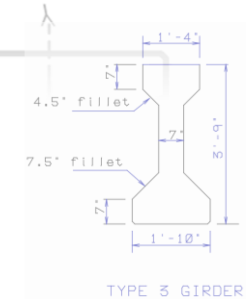


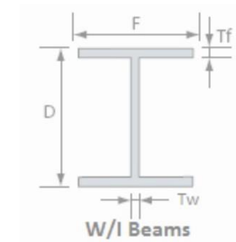


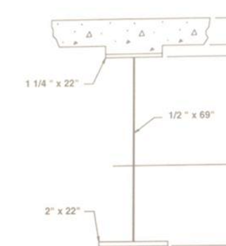


FIB-36

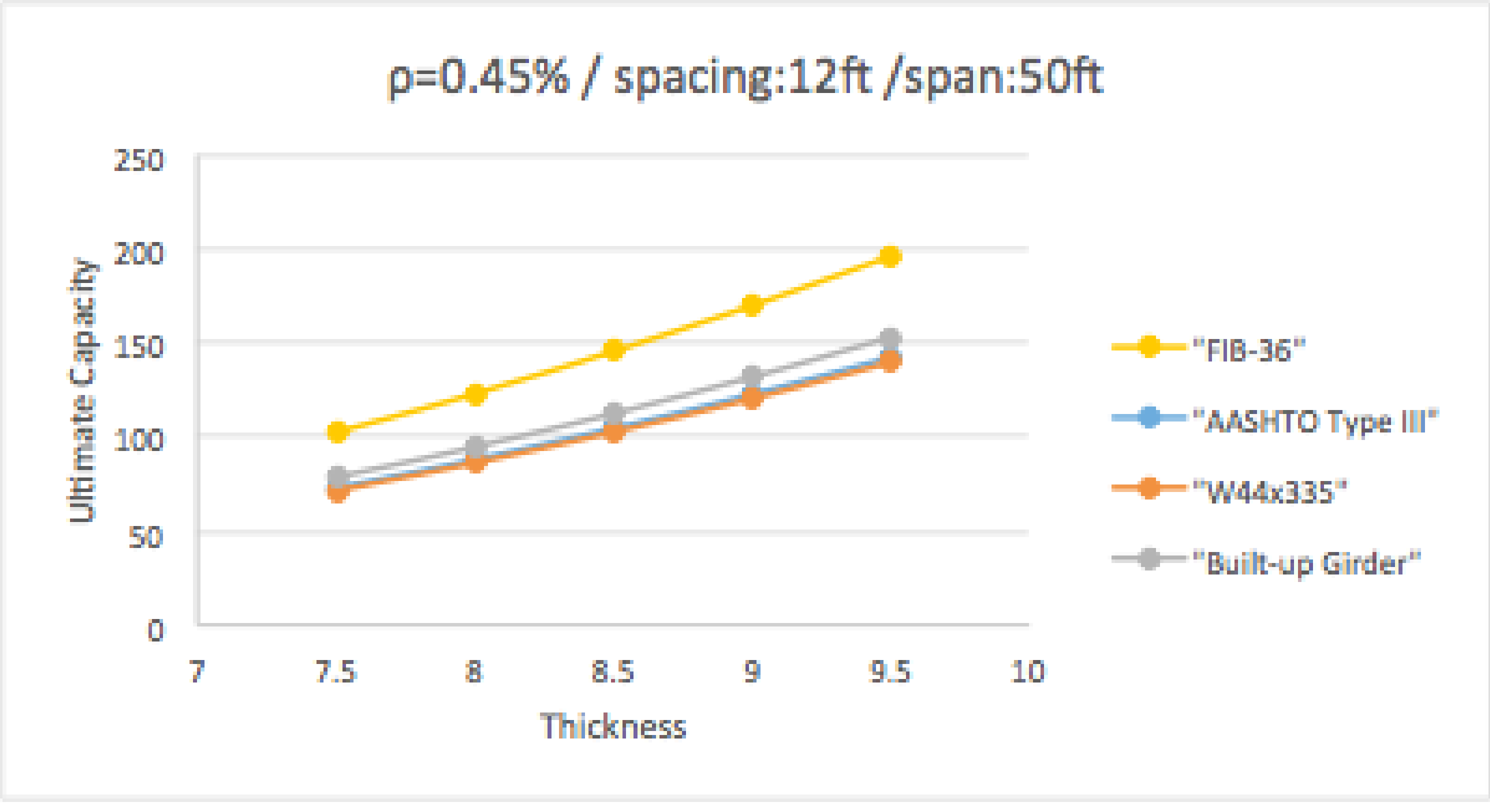
FIB-36
AASHTO TYPE III
W44x335
Built- up steel Girder



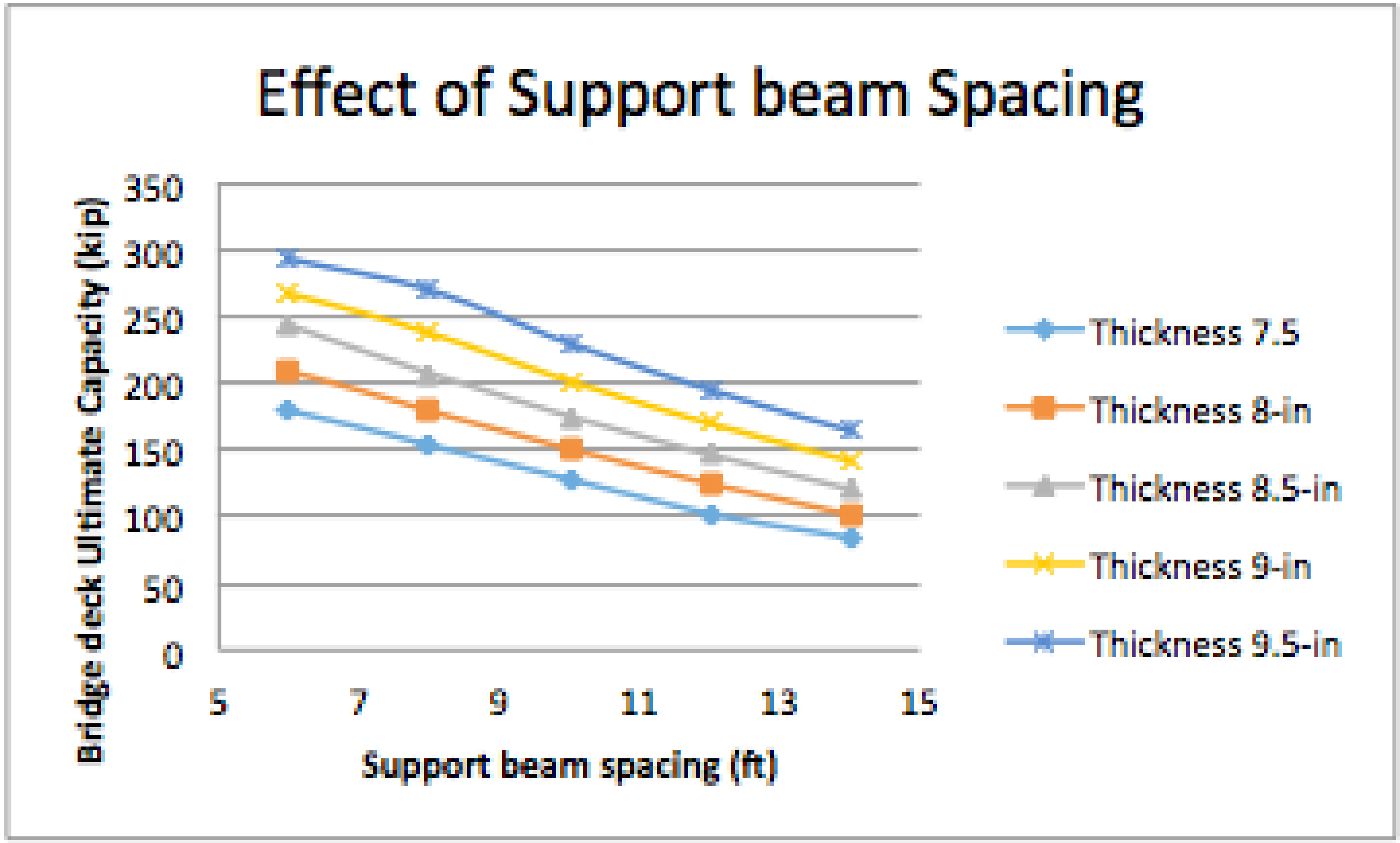


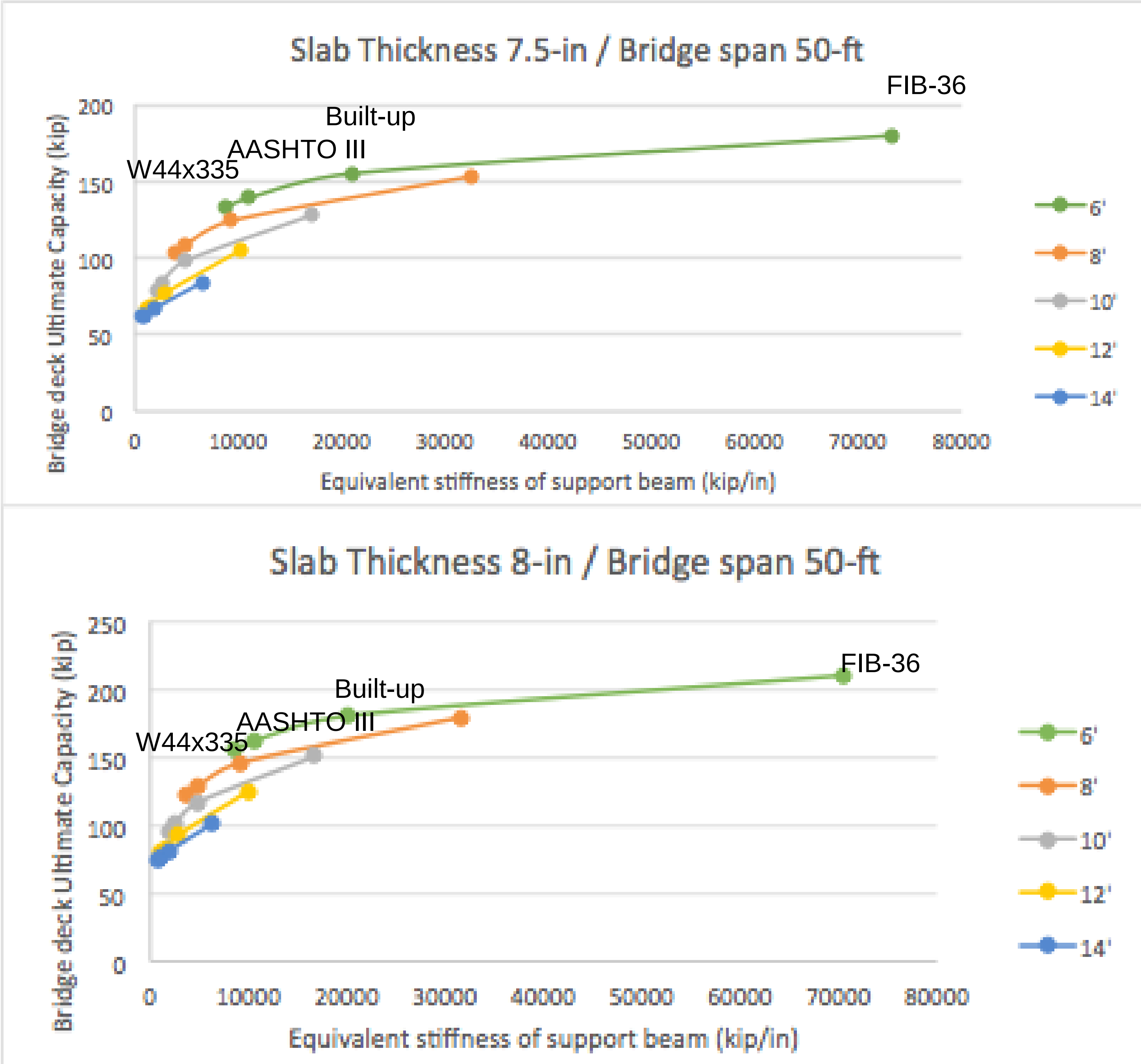


EFFECT OF DIFFERENT SUPPORT BEAMS

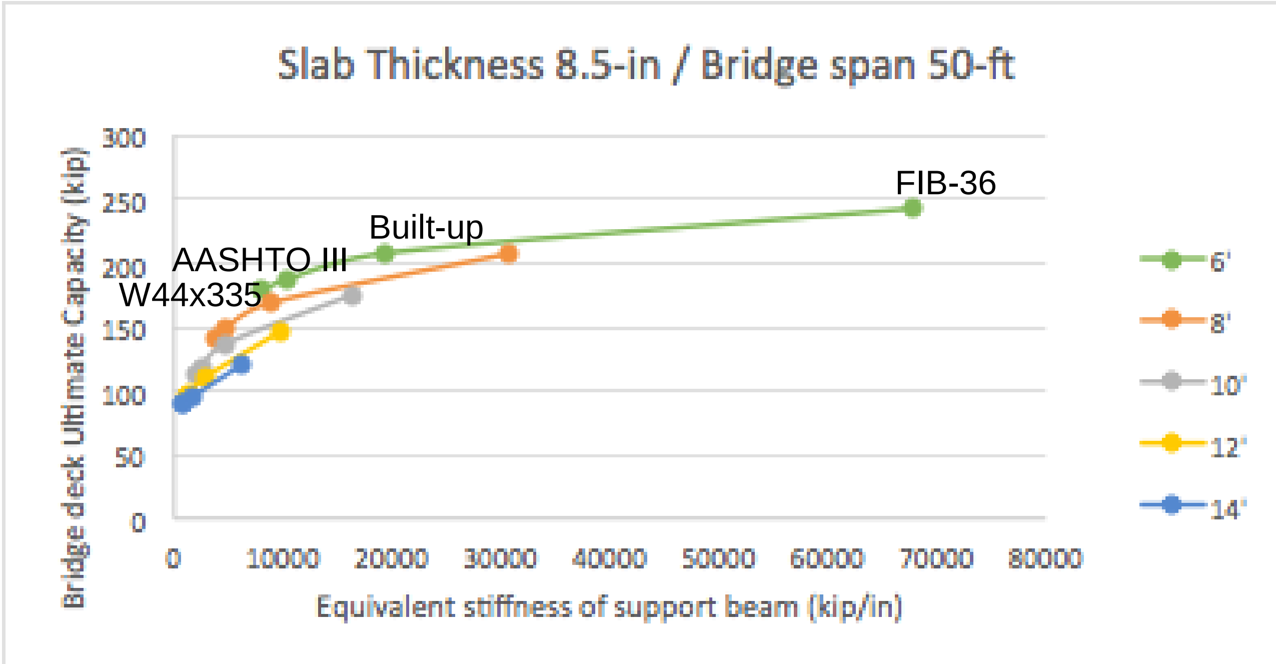


Effect of support beam spacing

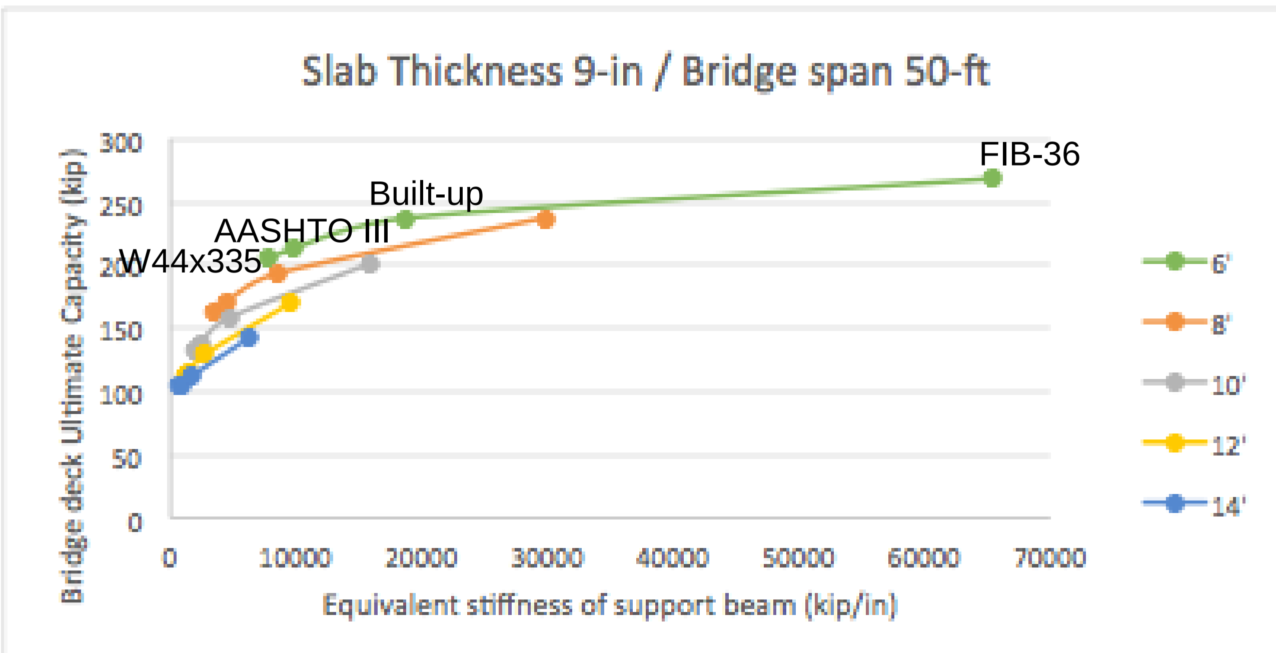




effect of lateral stiffness on ultimate load capacity



Slab Thickness 8.5-in / Bridge span 50-ft
Bridge deck Ultimate Capacity (kip)
300
250
200
150
100
50
0
0
10000
20000
30000
40000
50000
60000
70000
80000
Equivalent stiffness of support beam (kip/in)
FIB-36
Built-up
AASHTO III
W44x335
6'
8'
10'
12'
14'



Slab Thickness 9-in / Bridge span 50-ft
Bridge deck Ultimate Capacity (kip)
300
250
200
150
100
50
0
0
10000
20000
30000
40000
50000
60000
70000
Equivalent stiffness of support beam (kip/in)
FIB-36
Built-up
AASHTO III
W44x335
6'
8'
10'
12'
14'

Conclusions:

- The predicted ultimate capacity estimated using ACI 318-05 and BS5400 methodologies remained constant or slightly changed when varying the support beam spacing. However, the ultimate capacity calculated by BD81/02 and the TRC approach changed.

- The increase of concrete compressive strength (f’c) using the TRC approach had an effect on the ultimate capacity of the bridge deck slab.

- Using the TRC approach it was observed that when considering a small spacing, an increase of the steel reinforcement ratio would give a proportional increase to the flexural punching capacity. Yet, the ultimate capacity is the lesser of that flexural punching and shear punching capacities.

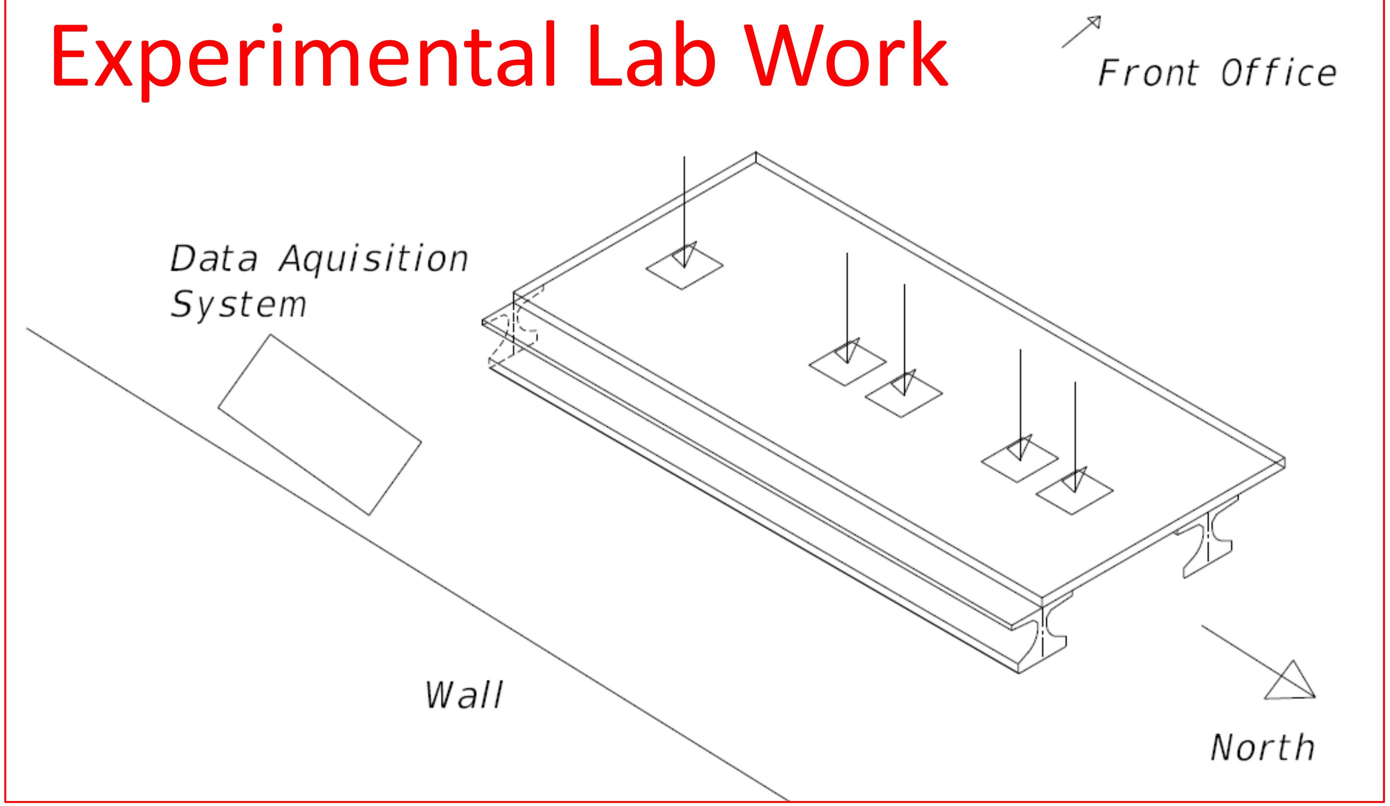
- Varying the bridge span length under fixed supporting beam spacing had little to no impact on the ultimate bridge deck capacity.

- When increasing the support beam spacing under a fixed deck slab thickness, the deck ultimate strength decreases.

- The support beam lateral stiffness has a direct relationship with the ultimate capacity of the bridge deck slab.

- It was observed that the FIB-36 girder contributed to a higher lateral stiffness when compared to the other girders like AASHTO and steel girders.

Experimental Lab Work



REINFORCEMENT STRAIN GAUGES

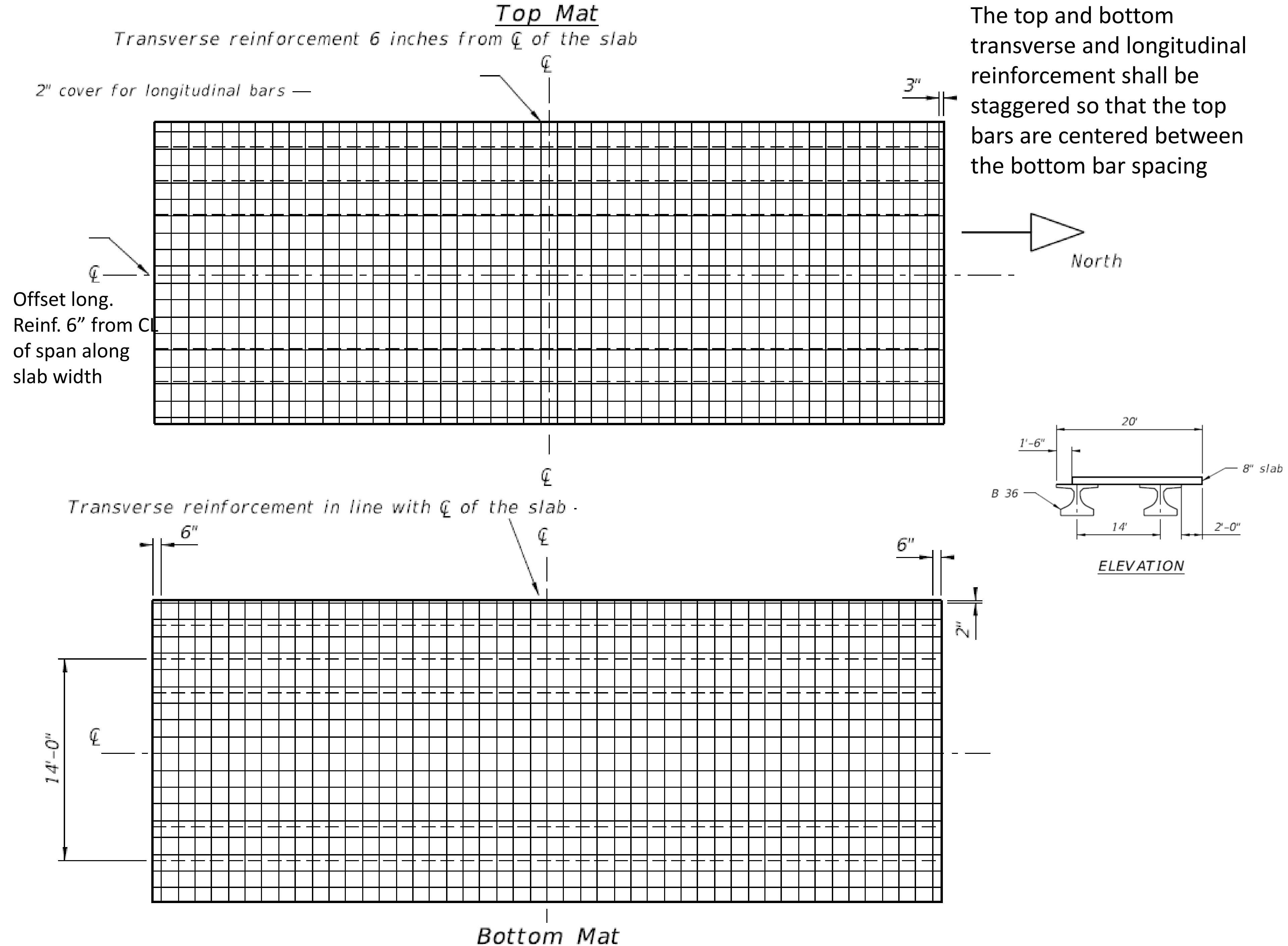
- ▪ 7 SG on bottom reinforcement in transverse direction (Along line of loading)
- ▪ 5 SG on bottom reinforcement in transverse direction (Parallel to line of loading) (1 ft away from load)
- ▪ 3 SG on bottom reinforcement in transverse direction (Parallel to line of loading) (3ft away from load)
- ▪ 1 SG on bottom reinforcement in transverse direction (Parallel to line of loading) (6 ft away from load)
- ○ 8 SG on top reinforcement in transverse direction
- ◇ 5 SG on bottom reinforcement in longitudinal direction

○ Top steel reinforcement strain gauge (Transverse)

▪ Bottom steel reinforcement strain gauge (Transverse)

◇ Bottom steel (Longitudinal)

□ Load location



Top Mat
Transverse reinforcement 6 inches from ℄ of the slab
2" cover for longitudinal bars
3"
The top and bottom transverse and longitudinal reinforcement shall be staggered so that the top bars are centered between the bottom bar spacing
North
Offset long. Reinf. 6" from CL of span along slab width
20'
1'-6"
8" slab
B 36
14'
2'-0"
ELEVATION
Transverse reinforcement in line with ℄ of the slab
6"
6"
2"
14'-0"
Bottom Mat

Concrete Cover:

- FHWA uses a 8” slab (Including the integral wearing surface).

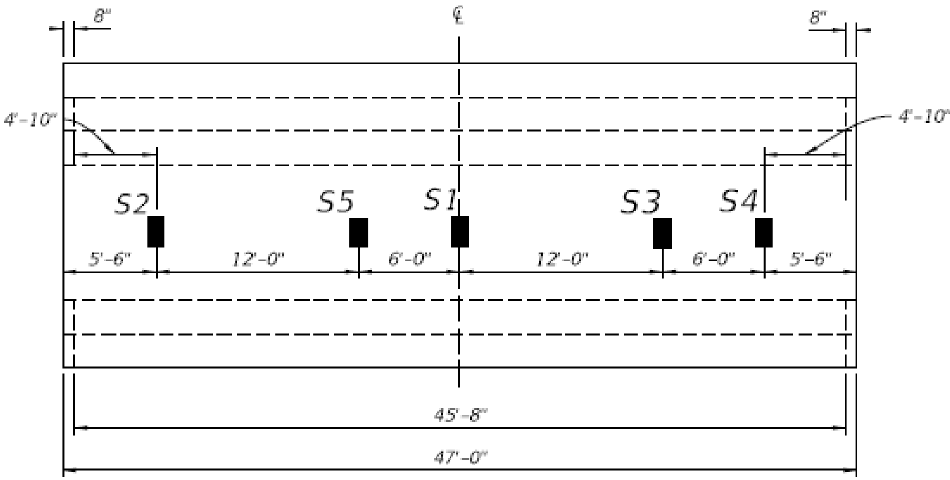
The integral wearing surface is considered in the weight calculations. However, for resistance calculations, the integral wearing surface is assumed to not contribute to section resistance, i.e., the section thickness for resistance calculations is assumed to be 7.5 in.

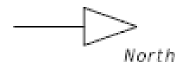
- FDOT SDG uses similar covers and same requirement for decks. The top cover may vary if it is a long or short bridge. Other than that, for decks, covers do not vary by environments.

	Slab Thickness	Top cover	Bottom cover
FHWA	8”	2.5”	1”
FDOT	8”	2”	2”

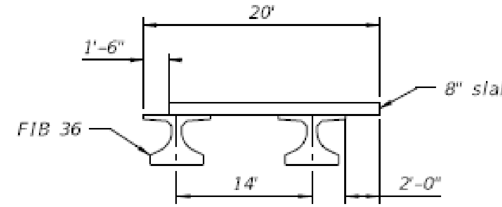
(These covers are measured from the extreme fiber of the slab to the steel surface)

- **For our 8” thick deck, 1.5” top cover and 2” bottom cover are used.**





Service Load Cases



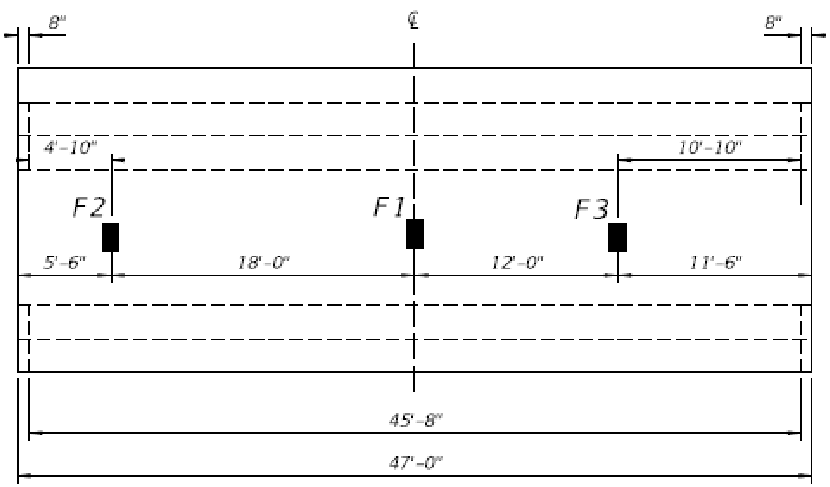
ELEVATION

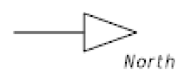
Empirical Design Method Top and Bottom Reinforcing Steel				
Span Length	Beam Spa.	Overall width	Transv. Reinf.	Long. Reinf.
45'-8"	14'-0"	20'-0"	No.5 at 12"	No.5 at 12"

Notes: Start testing 3 main service load cases S1, S2, S3 up to service load. The remaining service cases S4 through S5 are tentative based on the extent of damage from previous loads. If damage is minor continue to test S4 to S5. If damage is extensive, move on to failure load cases.

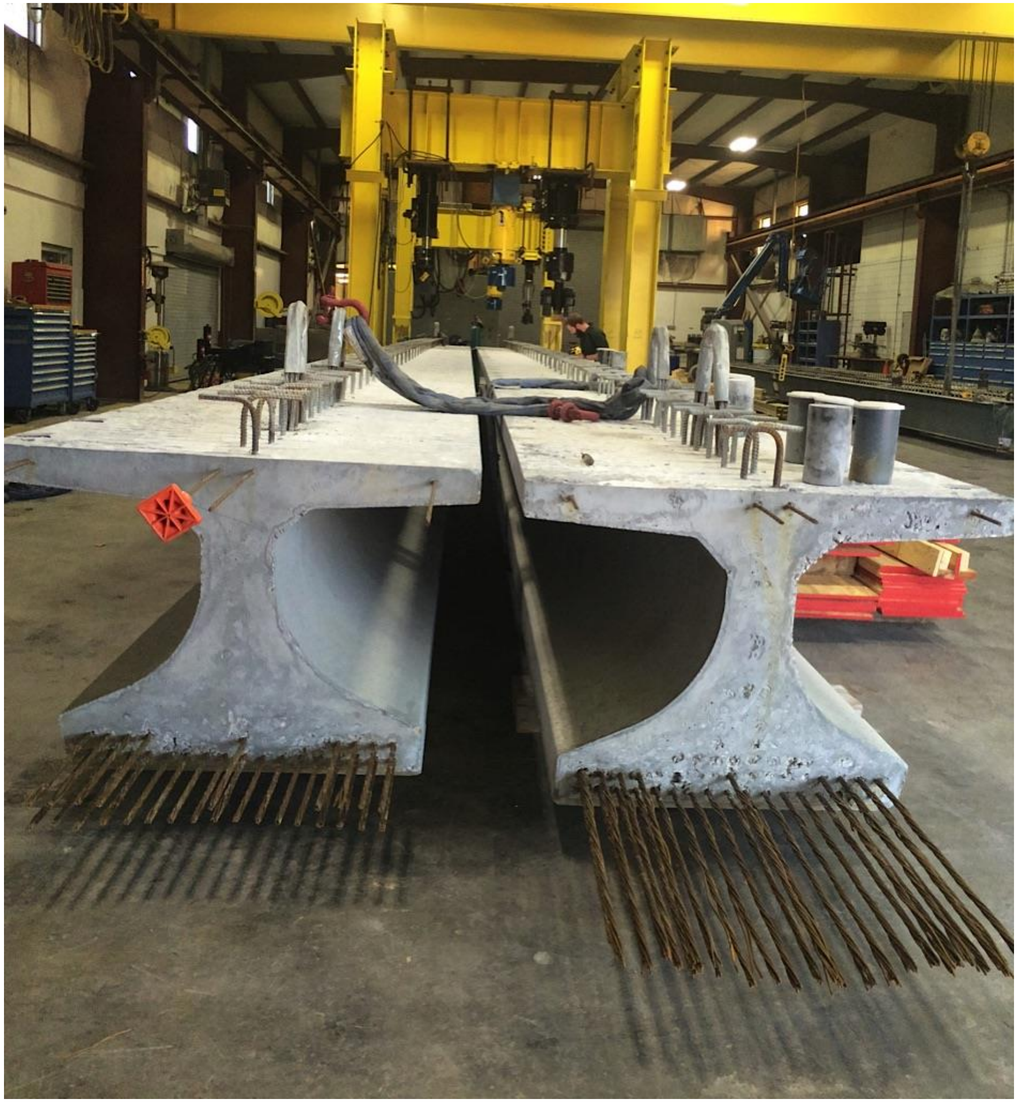
Deck will be cast to simulate real life scenario.

Load controlled loading



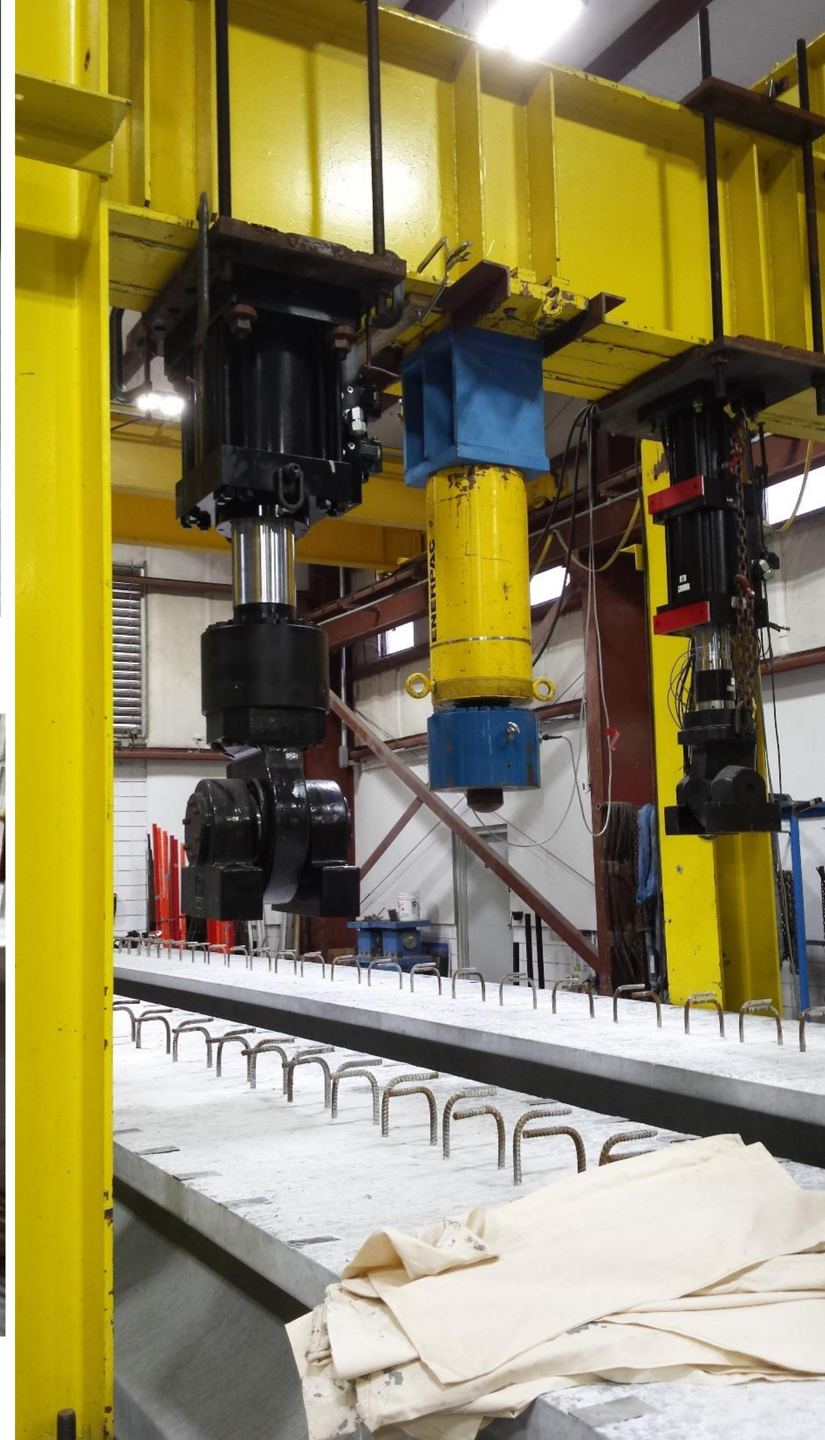


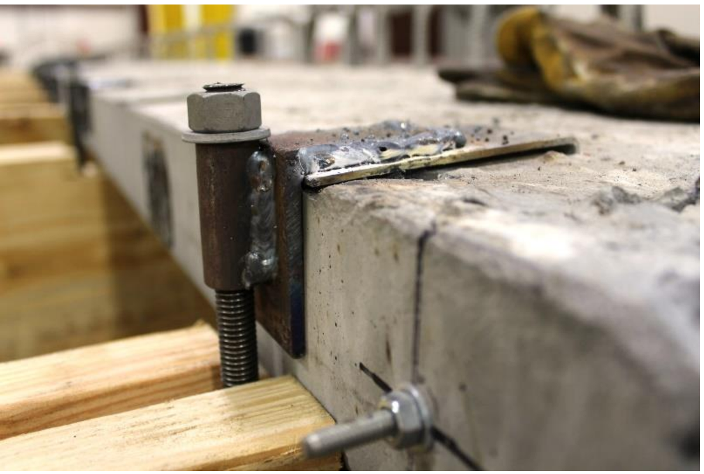
Failure Load Cases



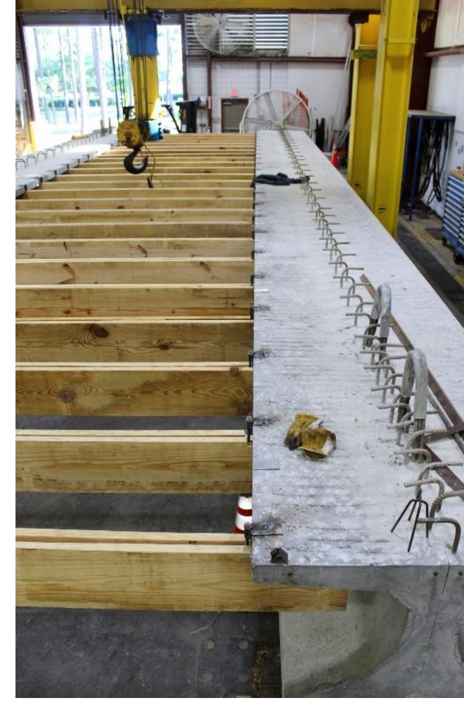
- Two FIB36 beams of 47 ft long.
- The composite action is attained by extending reinforcing stirrups from the top of the beams into the slab

hanger brackets embedded into the top flange of the FIBs

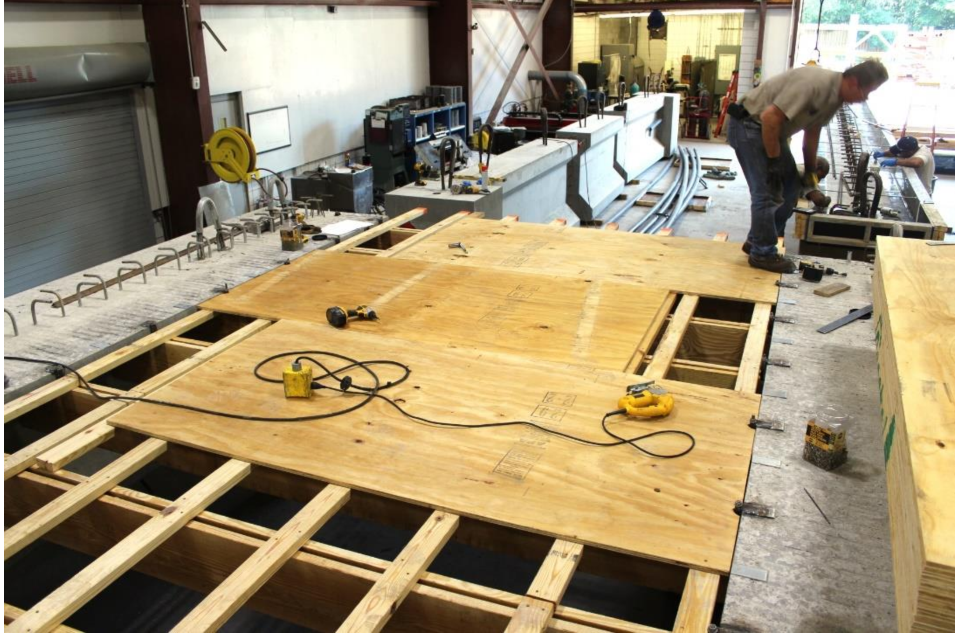

ENERPAC




NO.2
SYP 401



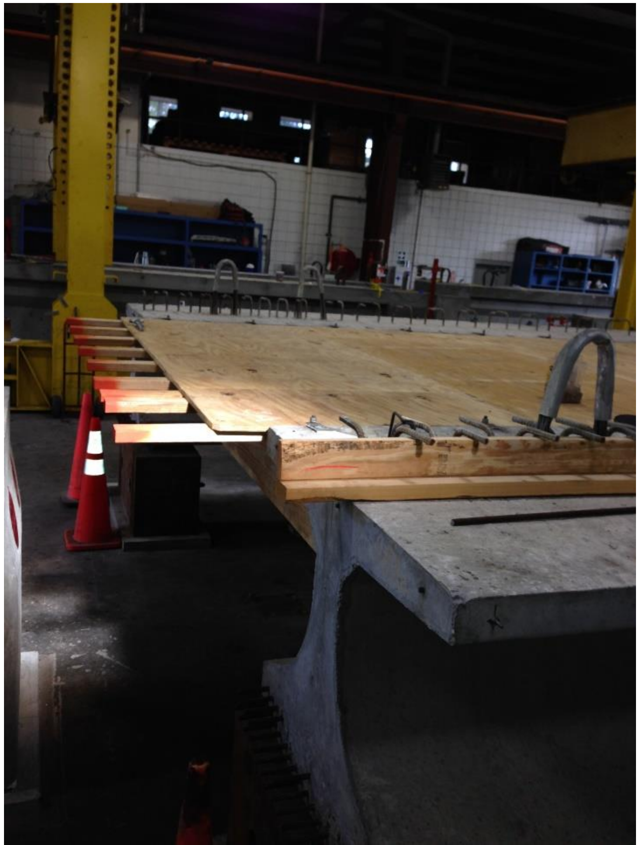

20-TON
Yale

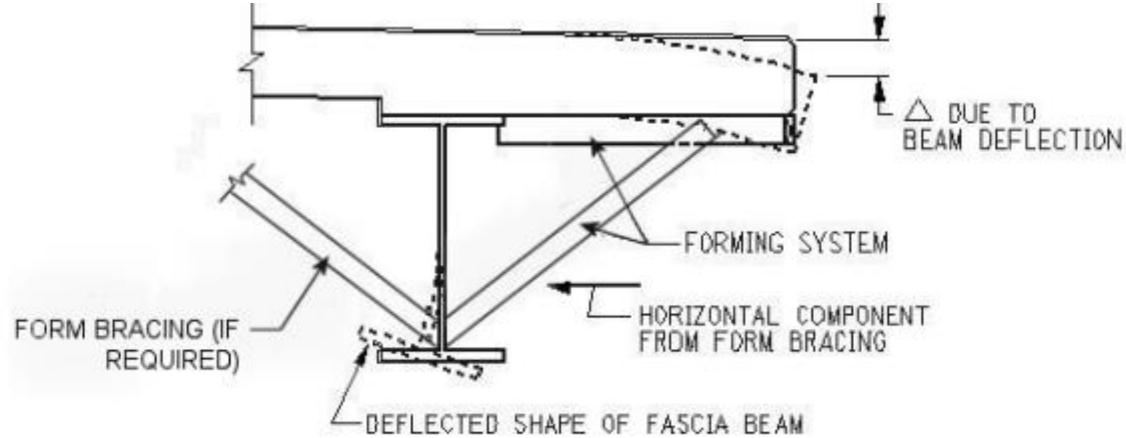



35 TON CAP
20 TON


R.J. MACK COMPANY
20 TON

R.J. MACK COMPANY





Overhang Form Bracing

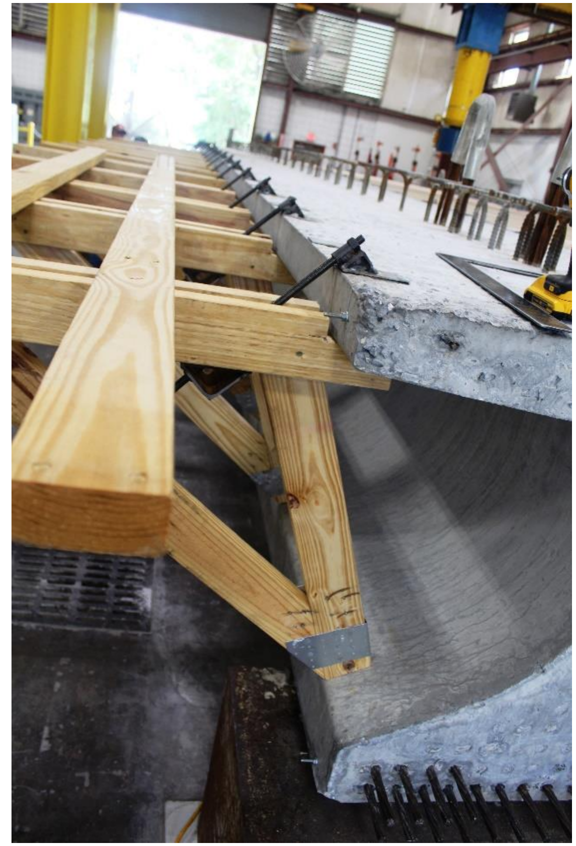
- FIB36 with form bracing: the horizontal force from the brace does NOT buckle the web, since it isn't bearing against the web of the fascia girder

- Forming and bracing systems used to place the concrete for bridge decks with large overhangs induce large horizontal forces in the fascia girder. These forces can cause lateral buckling and deflection problems in the fascia girder resulting in a poor deck profile.

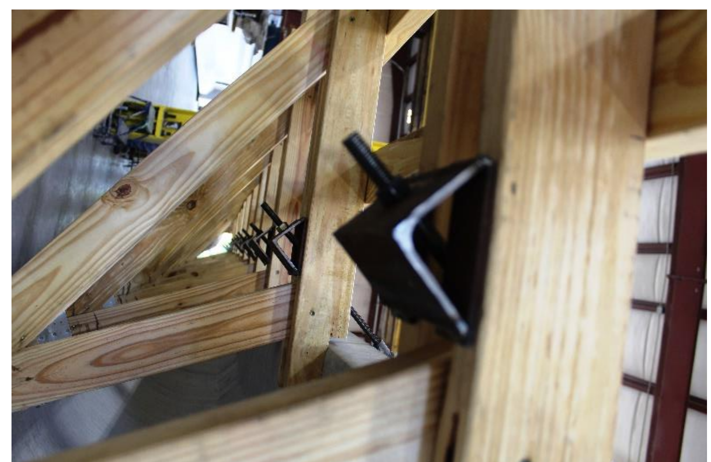
- The design evaluates the ability of the fascia beam to safely support the construction loads (including the forms, bracing, wet concrete, walkway overhangs, workforce, and concrete screeding machines and appurtenances.

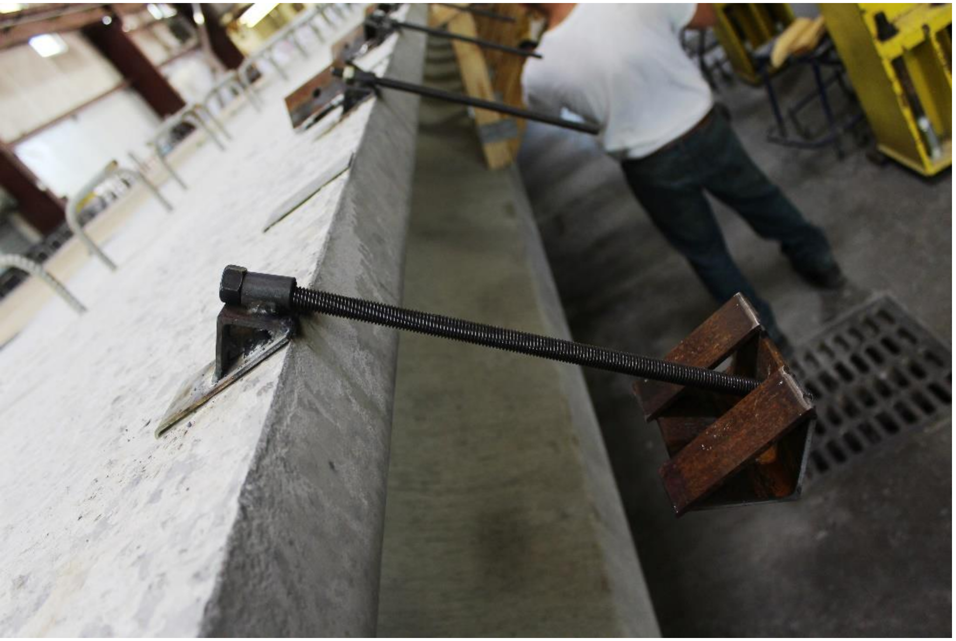
20-TON
South


ENERPAC
20-TON

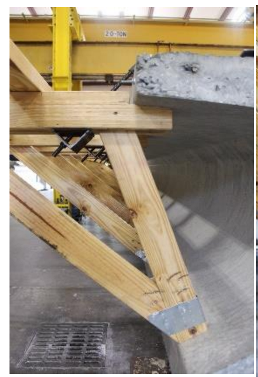



20-TON
South










CORNELL


20-TON



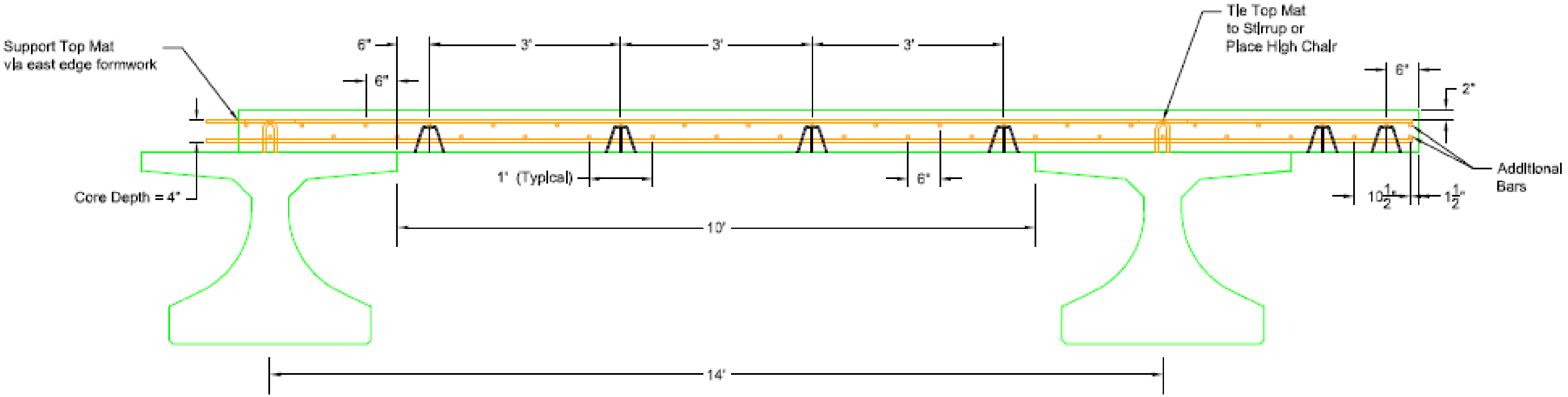

ENERPAC
CORNELL


ENERPAC

drawing showing the high chairs supporting the top mat

East

West

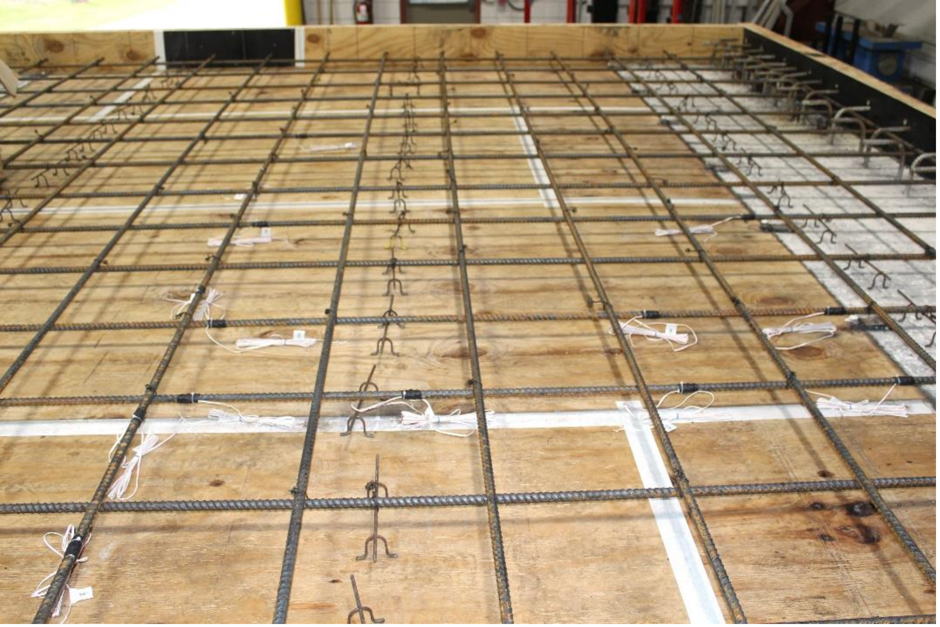


- Individual Upper Mat High Chair

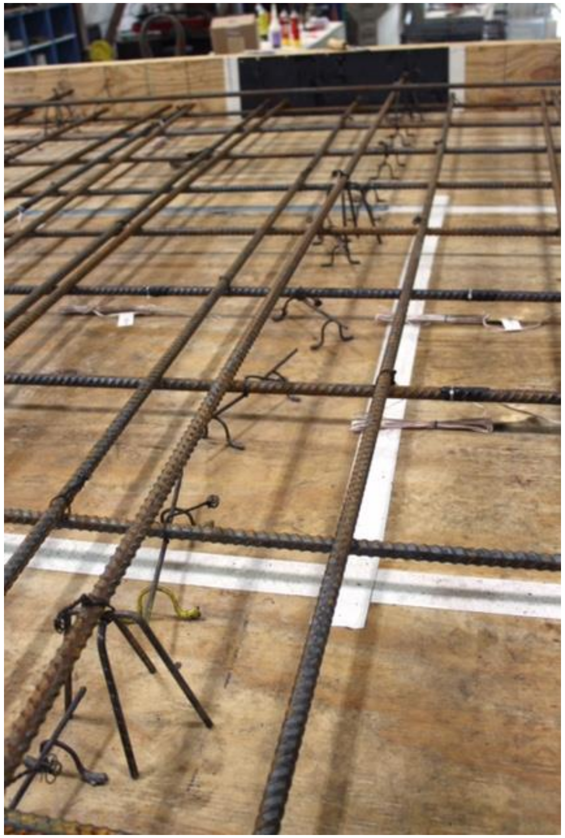
*Longitudinal (N-S) Spacing = 3'10"

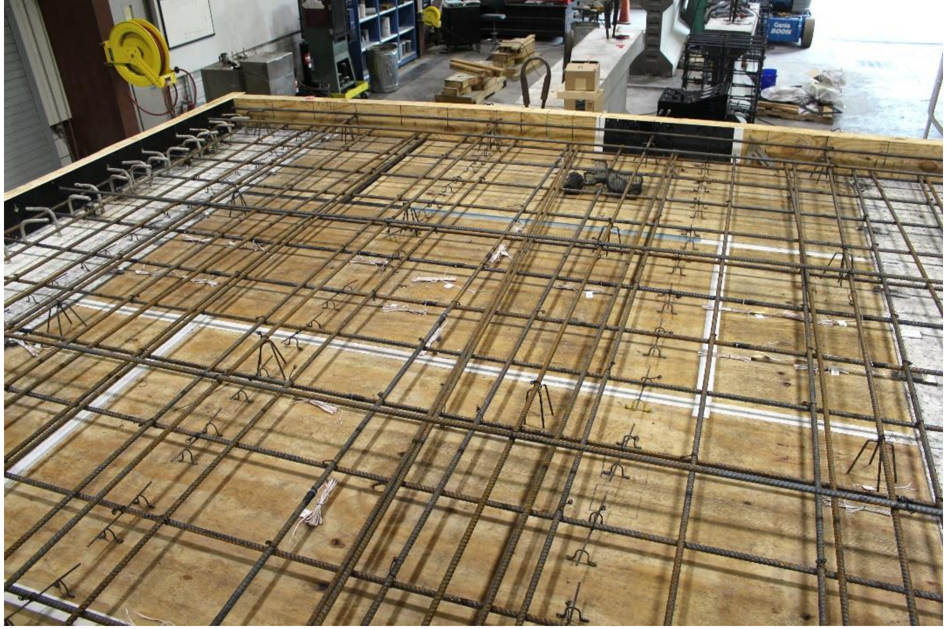
after first chair spaced 6" off N or S wall

Florida SDG suggests 2 inches of top and bottom cover for an 8 inch deck, we suggested using 1.5 inches for the top cover and 2 inches of bottom cover since our specimen does not include the wearing surface.

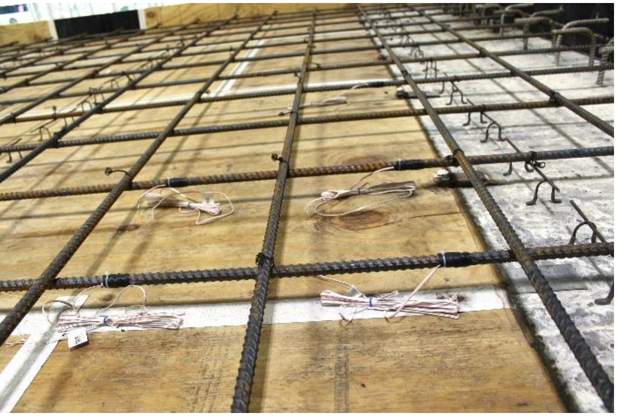
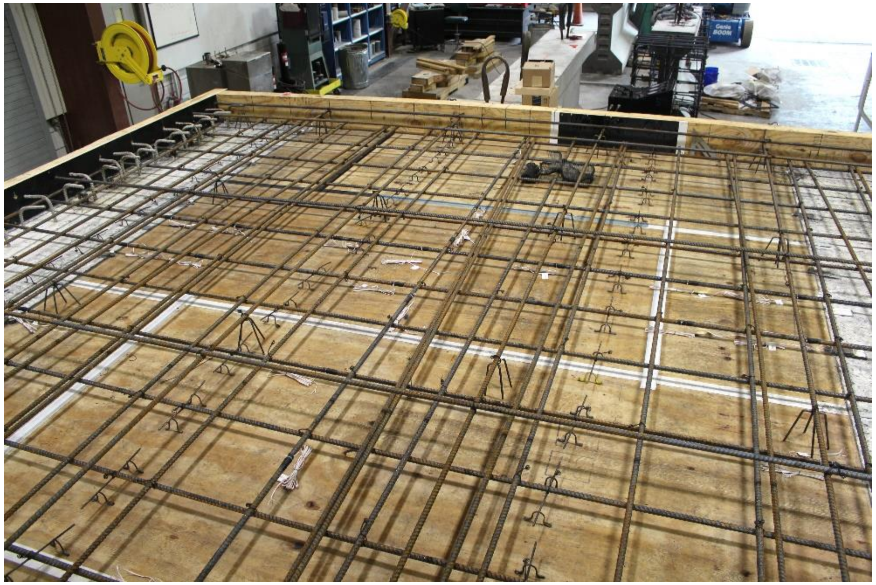




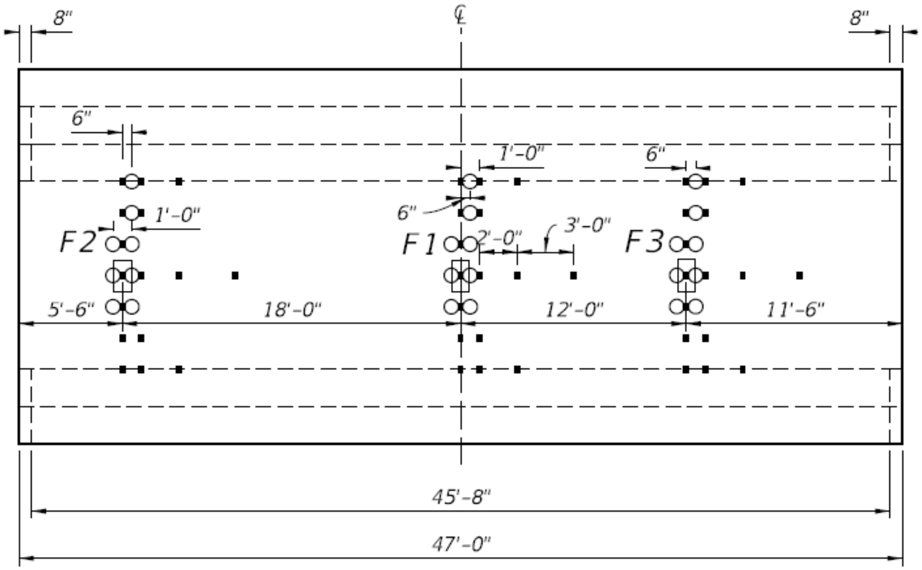




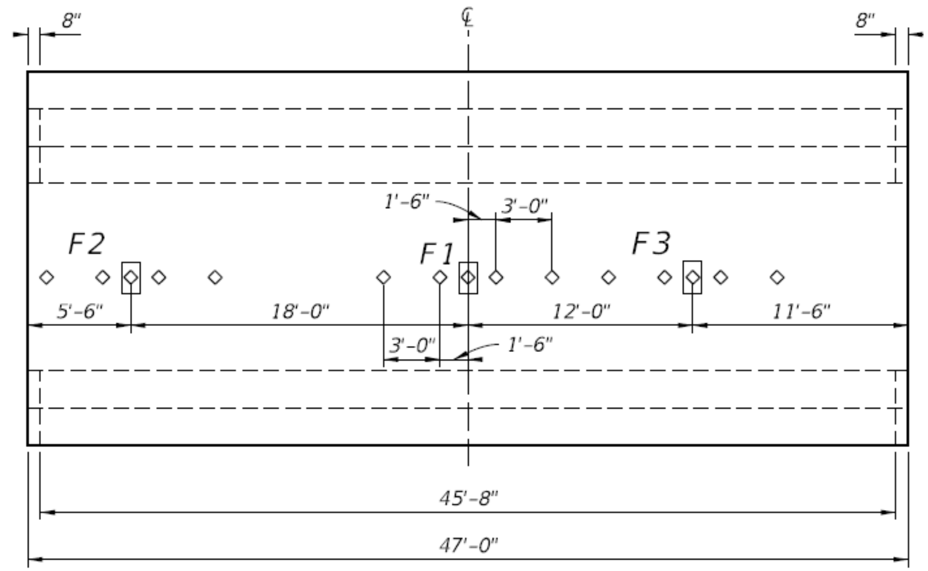


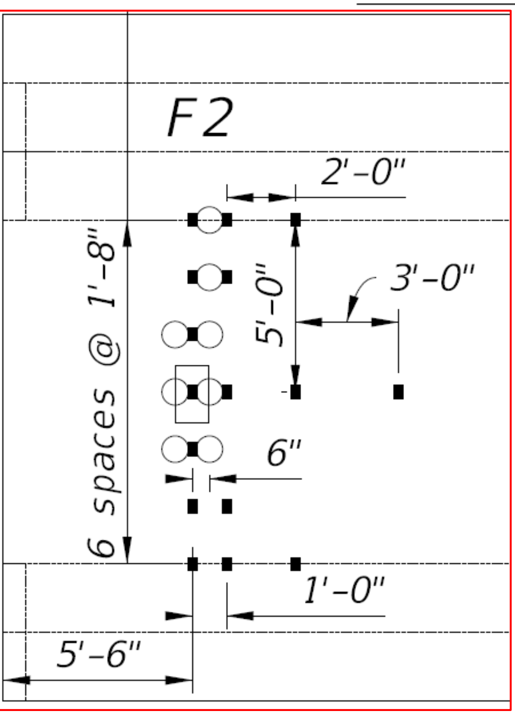
TRANSVERSE REINFORCEMENT STRAIN GAUGES

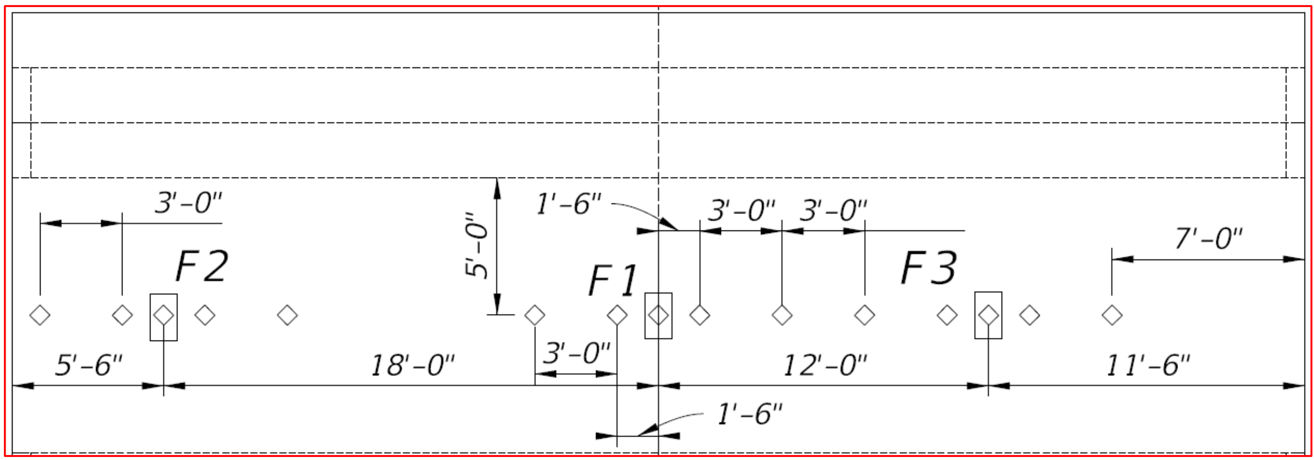


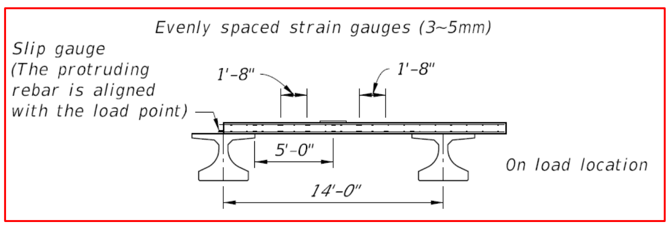
LONGITUDINAL REINFORCEMENT STRAIN GAUGES



Failure Load Cases







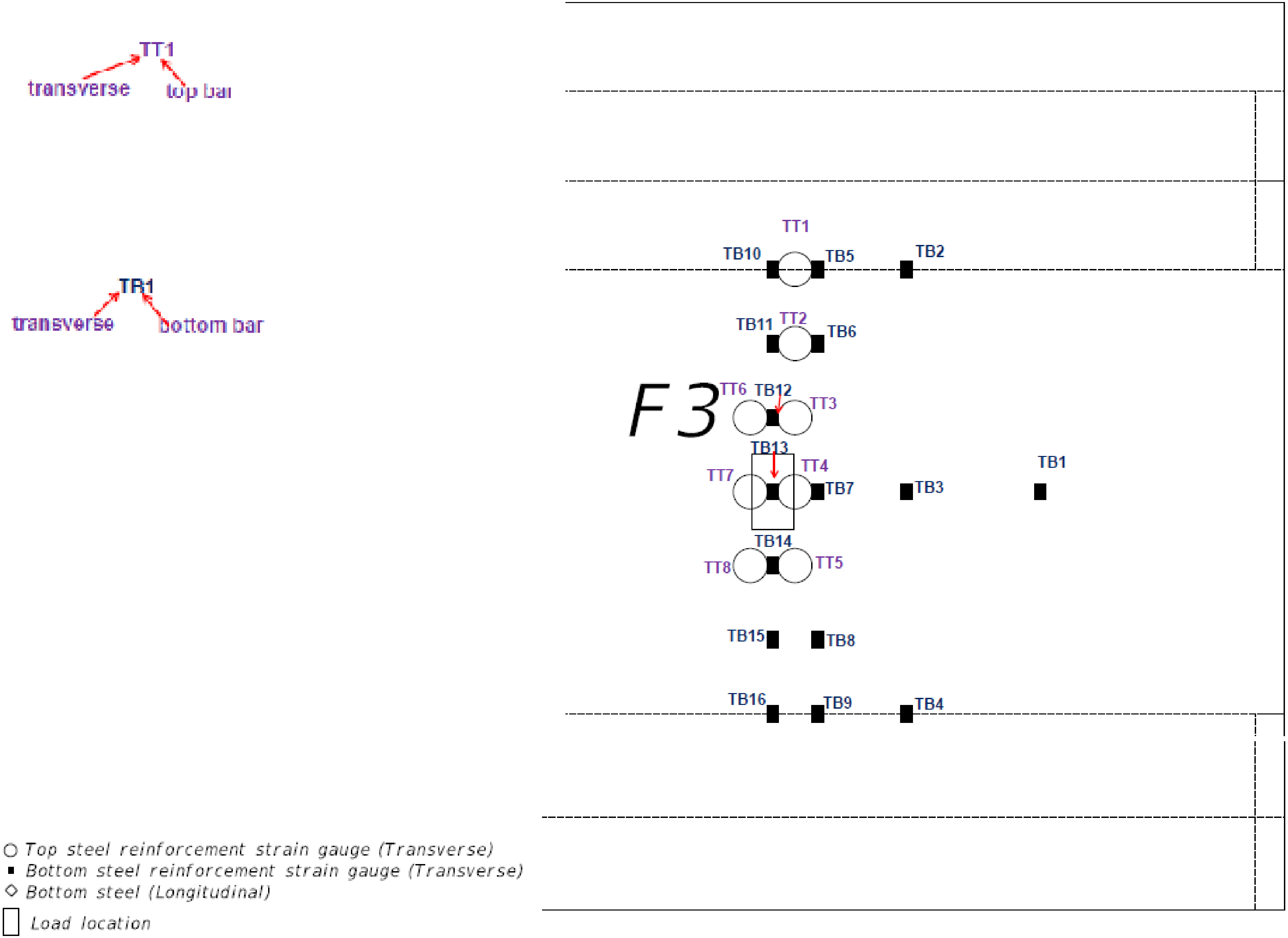
○ Top steel reinforcement strain gauge (Transverse)

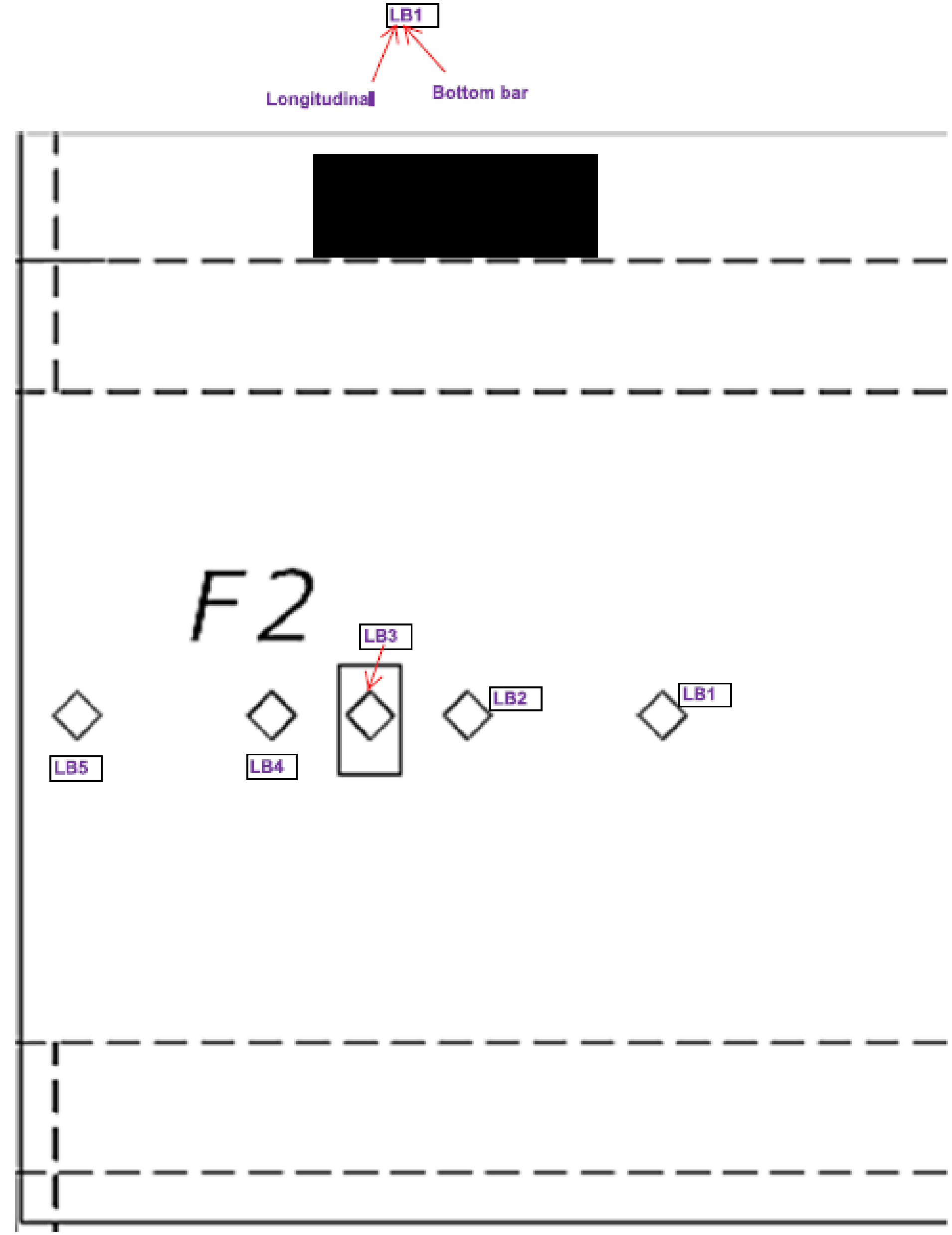
▪ Bottom steel reinforcement strain gauge (Transverse)

◇ Bottom steel (Longitudinal)

▯ Load location

Strand slip gauges & Crack gauges & LVDTs, SGs,


TT1
transverse
top bar
TB1
transverse
bottom bar
TT1
TB10
TB5
TB2
TB11
TT2
TB6
F3
TT6
TB12
TT3
TB13
TT4
TT7
TB7
TB3
TB1
TB14
TT8
TT5
TB15
TB8
TB16
TB9
TB4
Top steel reinforcement strain gauge (Transverse)
Bottom steel reinforcement strain gauge (Transverse)
Bottom steel (Longitudinal)
Load location



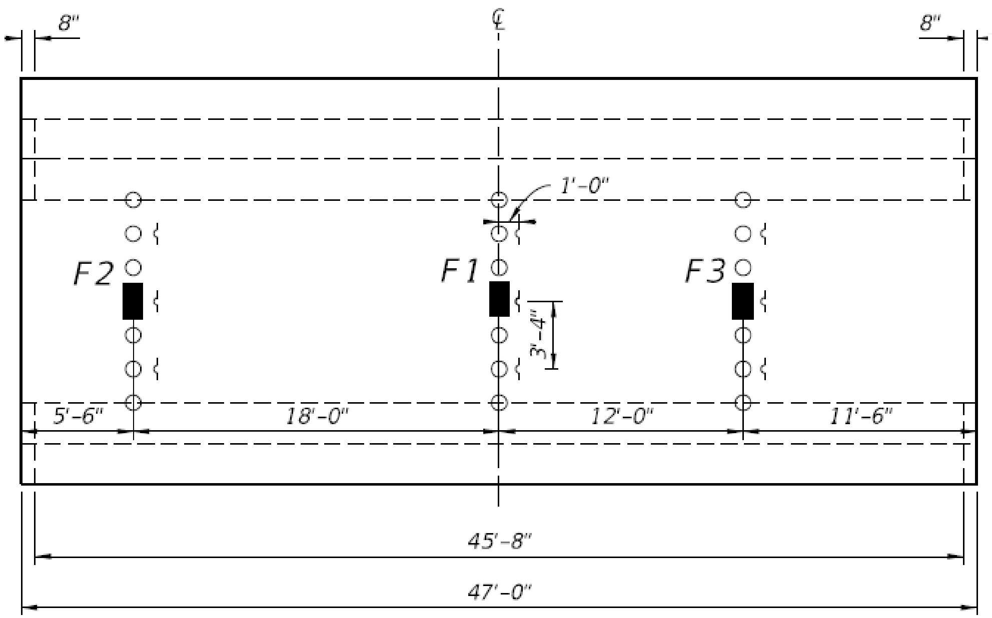
○ *Top steel reinforcement strain gauge (Transverse)*

▪ *Bottom steel reinforcement strain gauge (Transverse)*

◇ *Bottom steel (Longitudinal)*

▯ *Load location*

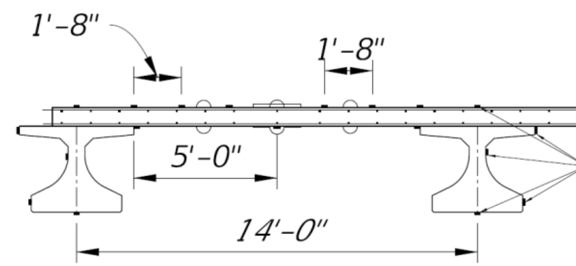
Top concrete slab strain gages & crack gages



Concrete top slab strain gauges & crack gauges

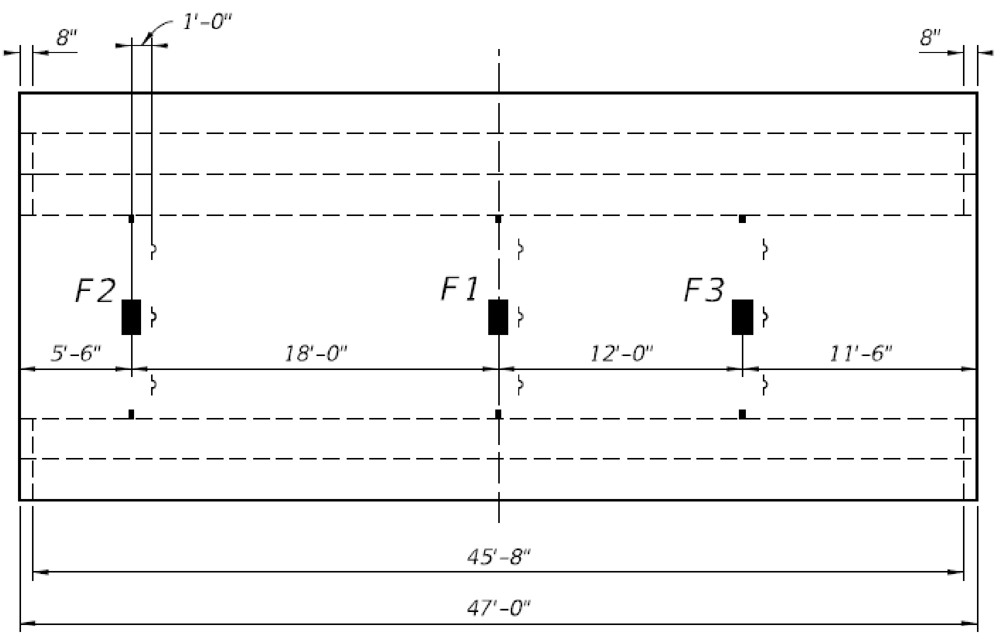
○ Top slab strain gauge (Transverse) (Foil) (60 mm)
} Full bridge crack strain gauge (Transverse) (200 mm)

6 Strain gauges (foil) on top of slab per loading location ~ 18 total for slab
3 full bridge crack strain gauges on top of slab per loading location



*Install 4 strain gauges per beam on one side as shown and an extra foil strain gauge on top of the deck, above the centerline of the beam at midspan and near supports.

Bottom concrete slab strain & crack gauges



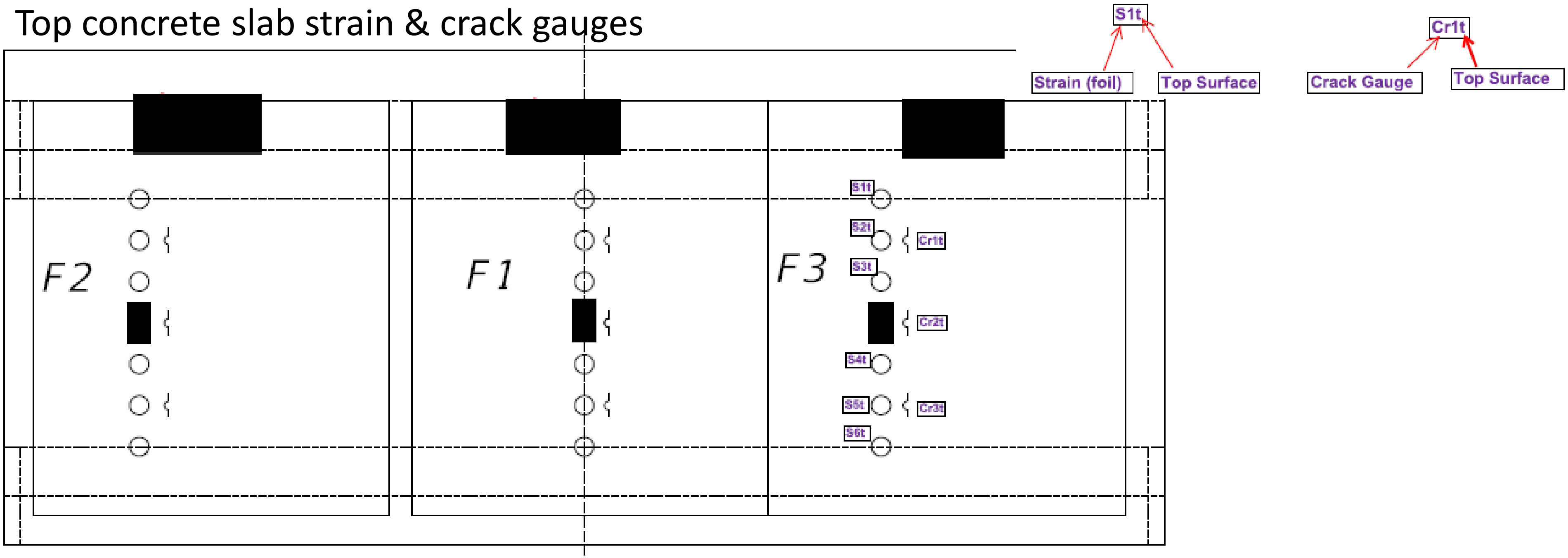
Concrete bottom slab strain gauges & crack gauges

▪ Bottom slab strain gauge (Transverse) (Foil) (60 mm)
} Full bridge crack strain gauge (Transverse) (200 mm)

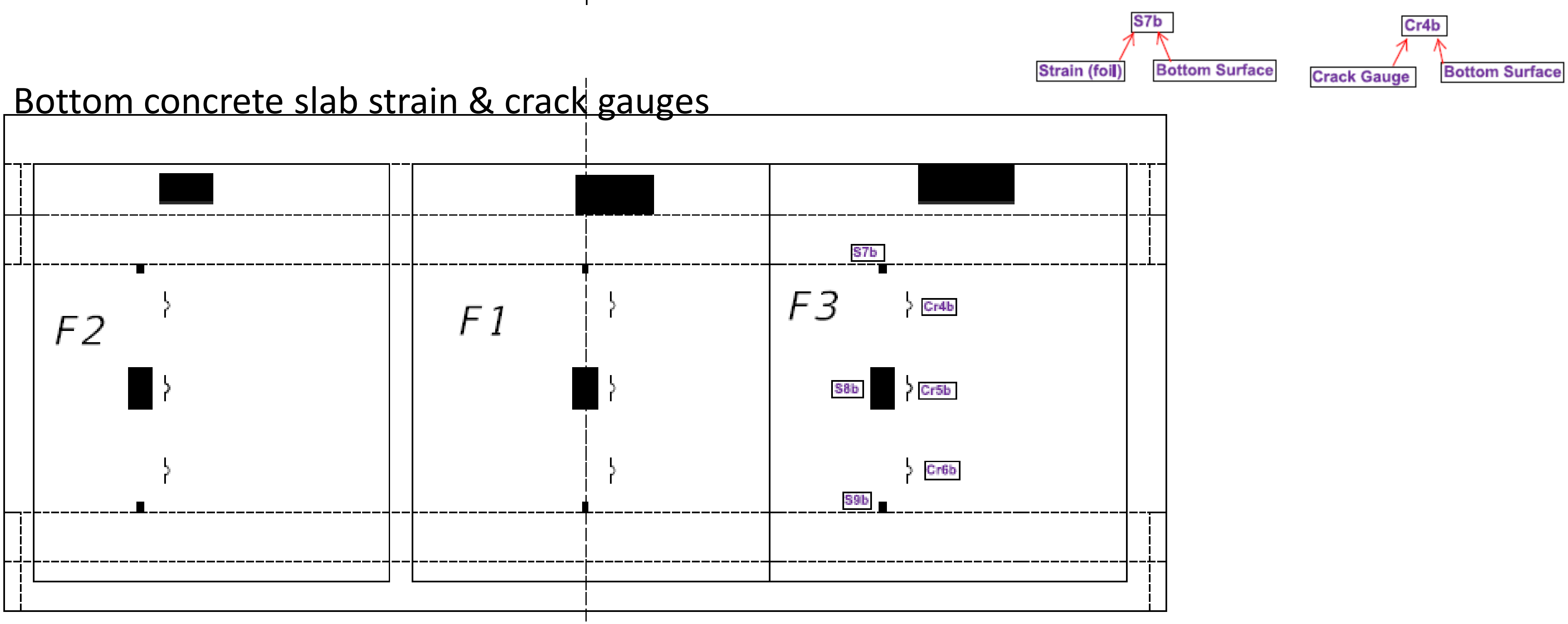
} 3 full bridge crack strain gauges on bottom of slab per loading location

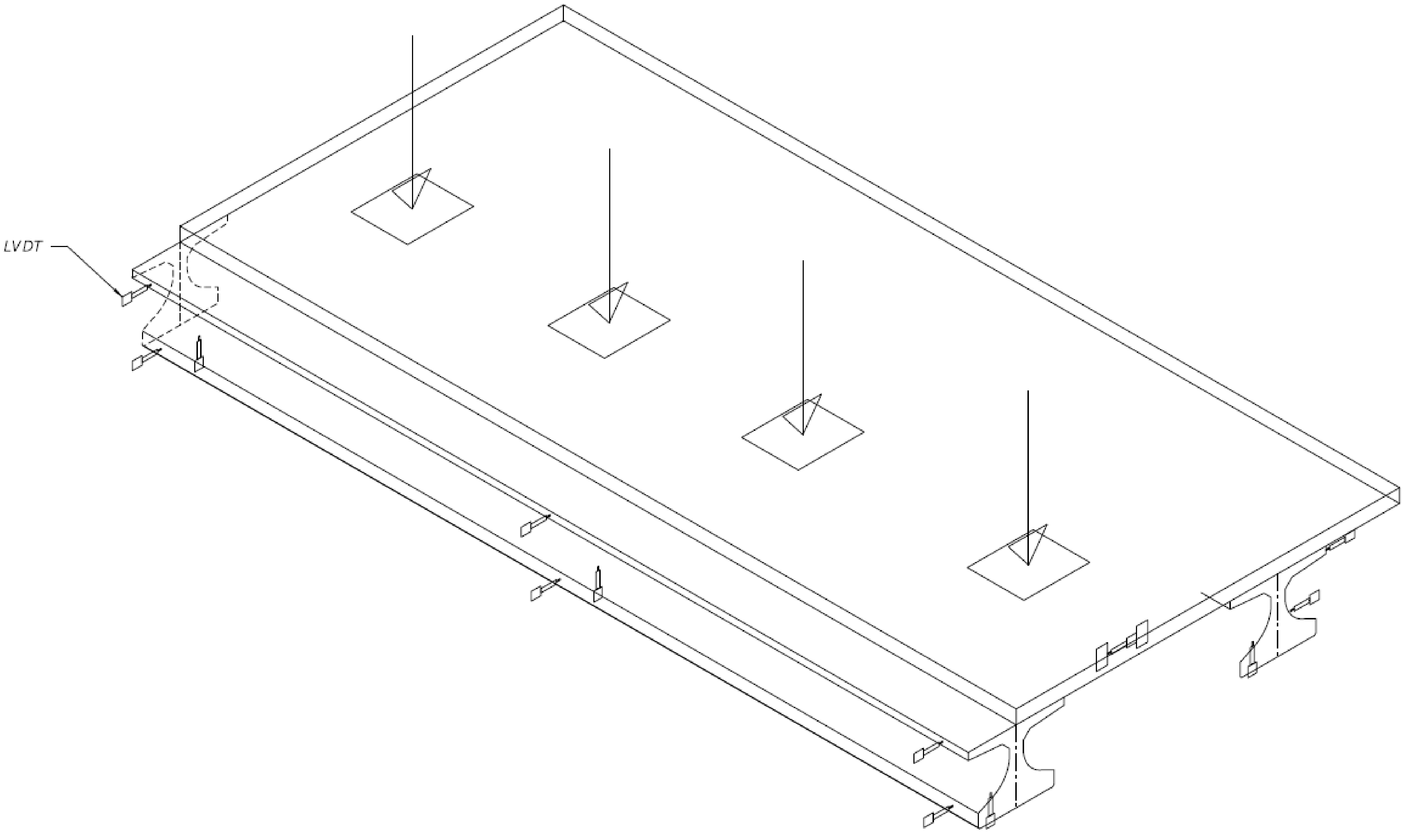
▪ 3 Bottom slab strain gauges on bottom of slab per loading location

Top concrete slab strain & crack gauges

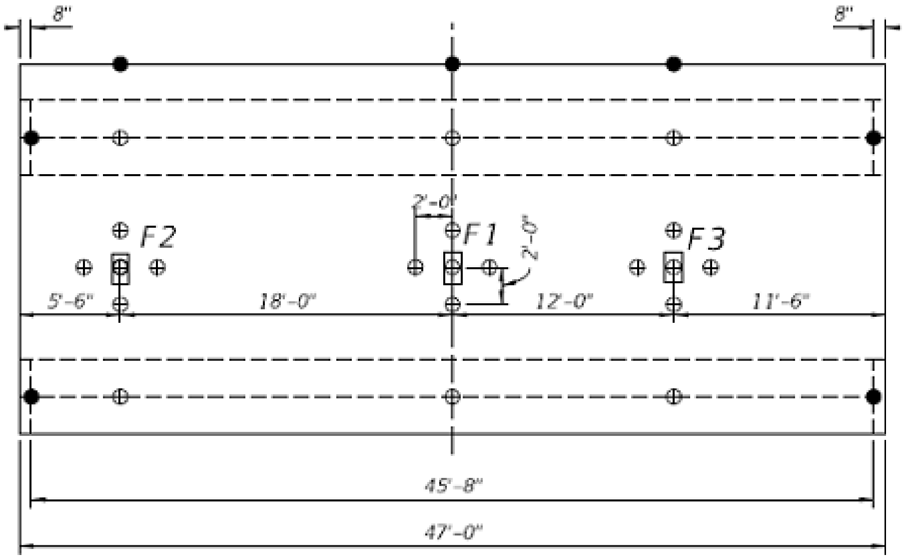


Bottom concrete slab strain & crack gauges




LVDT

Vertical Deflection Gauges



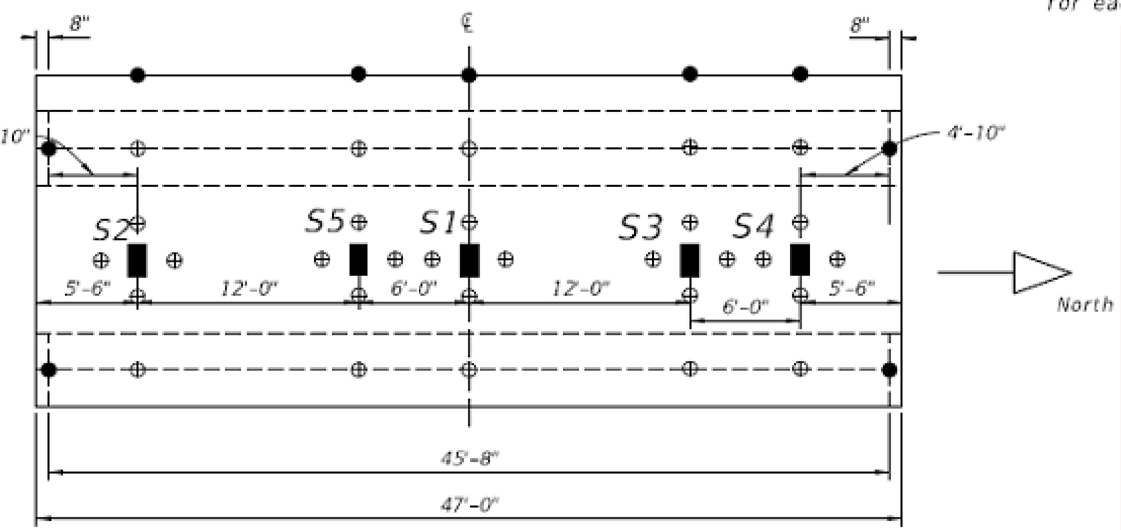
Failure Load Cases

● 1 Vertical deflection gauge on top of slab per load location

⊕ 7 Vertical deflection gauge below slab or beam per load location

● 4 Vertical deflection gauge 's on top of slab that are above the supports stay throughout testing

Therefore, 8 deflection gauge 's are movable, they can be moved to each test accordingly.

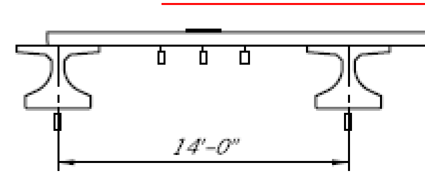


Service Load Cases

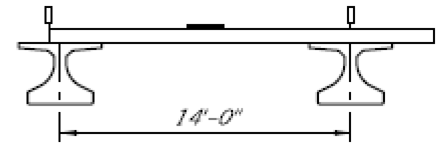
Vertical Deflection Gauges

● Vertical deflection gauge on top of slab

⊕ Vertical deflection gauge below slab or beam



(On failure load locations)
Showing vertical deflection gauge 's only

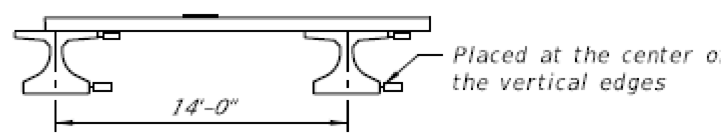


(On supports section)
Showing vertical deflection gauge 's only

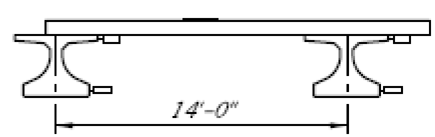
Elevation view (vertical deflection gauges only)

Horizontal Deflection Gauges

▭ 4 Horizontal deflection gauge 's total per load location. They can be moved for each test

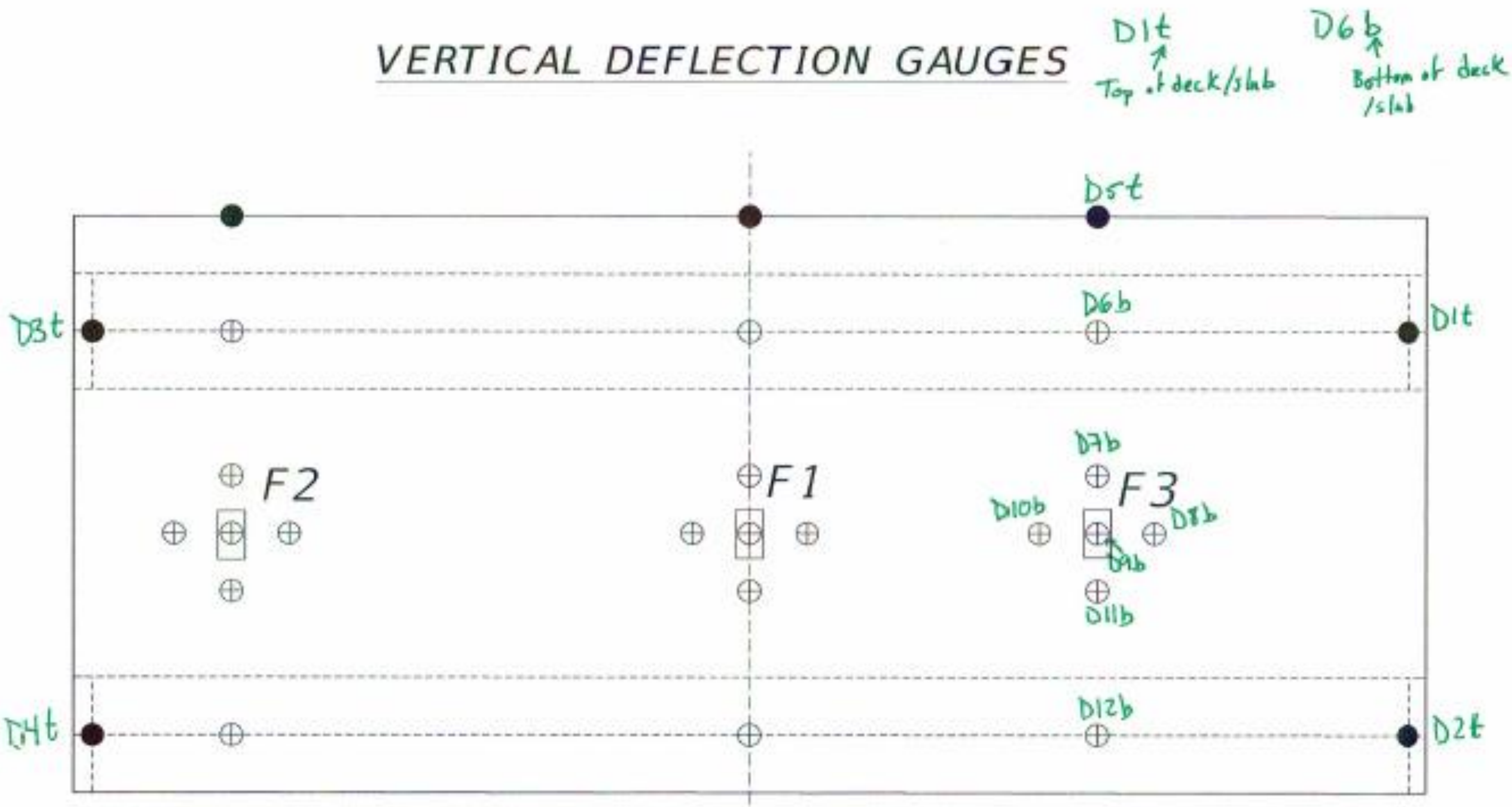


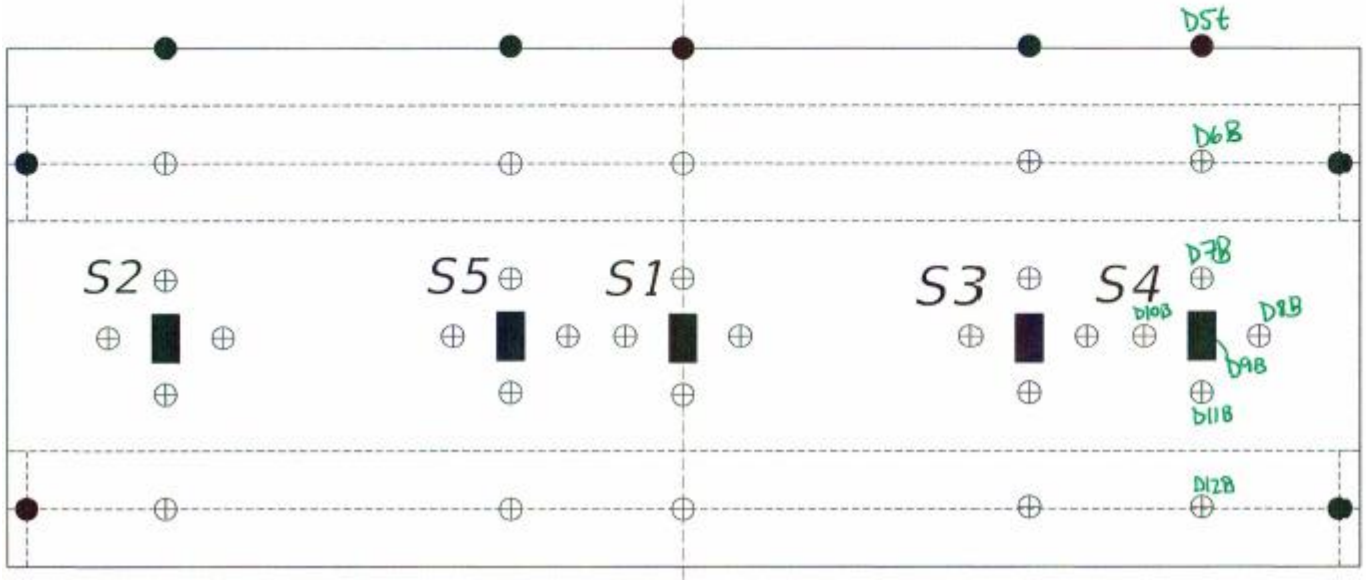
(On failure load locations)
Showing Horizontal Deflection Gauges only



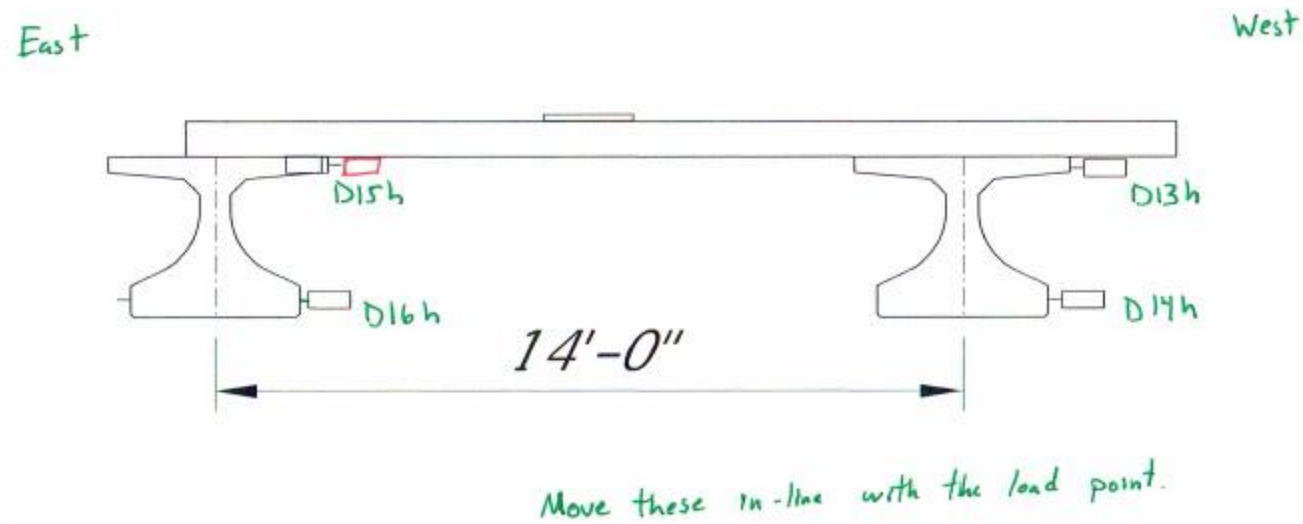
(On support locations)
Showing Horizontal Deflection Gauges only

VERTICAL DEFLECTION GAUGES




D5t
D6B
S2
S5
S1
S3
S4
D7B
D10B
D8B
D9B
D11B
D12B

Horizontal Deflection Gauges



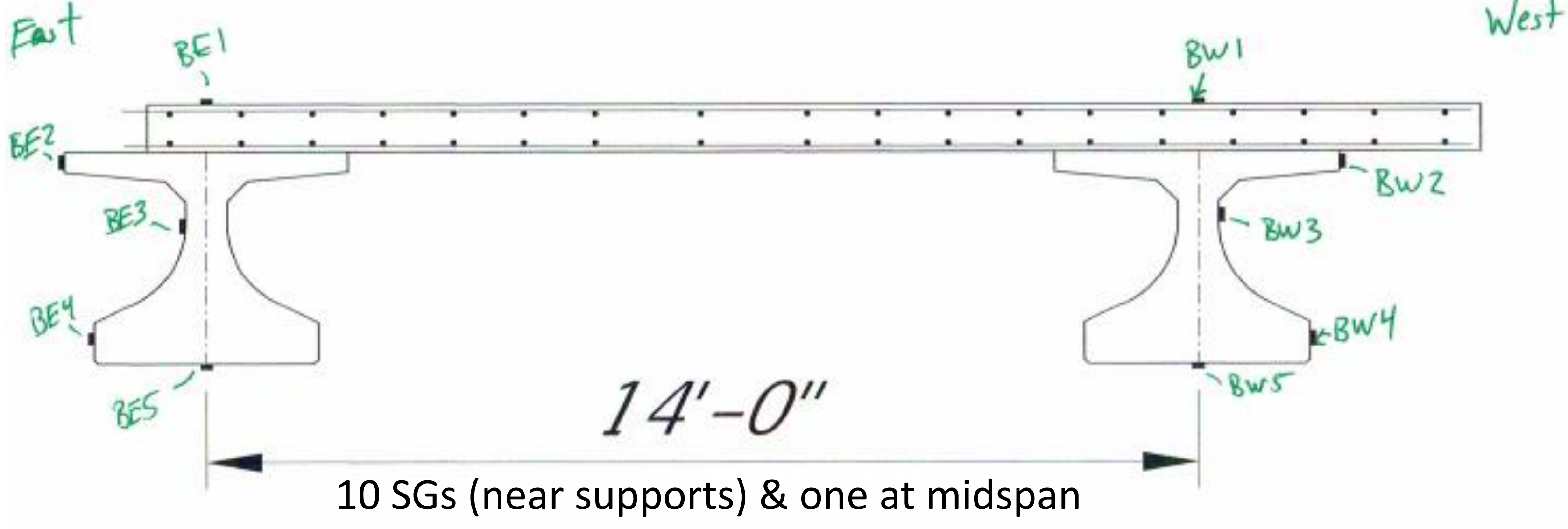
The support LVDTs stay in their original locations, the transverse (horizontal) and vertical gauges at the loading location should move to coincide with the loading location.

Move both vertical and horizontal deflection gauges (transverse to load) for each service load and failure load testing locations.

Deflection gauge s at the supports remain in place throughout testing

The testing will start with service loads starting from S1 to S2 to S3 to S4 to S5. (Move deflection gauge for each test)

Then, failure load cases for F1 to F2 to F3 in that order.



Proposed Strain Rosette on the web of the beam on each side of the beam to monitor shear strain (R1L, R1V & R1_45).

Challenge: the FIB 36s don't really have much of a flat web to easily put these gauges.

Precautions for Concrete Pour:

Continuous Structure Deck Slab Placements Deck slabs on continuous structures are subject to transverse cracking during construction. The cracking can be found in negative moment areas where the concrete has already set and the placement has continued into positive moment areas.

The cracking is caused by additional deflection of the beams when the concrete in the remaining positive moment area is placed. The frequency of the cracking can be reduced if proper construction methods are used and strict control over the timing and sequencing of the deck placement operation is exercised.

Avoid Deflection cracks by:

- Reducing the duration of placement (Avoid Slow Rate of Placement)
- Increasing the time to initial set of the concrete (use retarding admixture to assure that initial set will not occur prior to completion of the placement)

Questions?